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NATIONAL DAM SAFETY PROGRAM. SPLIT ROCK POND DAM (NJ00264), PAS--ETC(U)
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BEAVER BROOK, MORRIS COUNTY

PASSAIC RIVER BASIN

NEW JERSEY

SPLIT ROCK POND DAM

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

NJ 00264

DEPARTMENT OF THE ARMY
PHILADELPHIA DISTRICT, CORPS OF ENGINEERS
CUSTOM HOUSE - 2D & CHESTNUT STREETS
PHILADELPHIA, PENNSYLVANIA 19106
AUGUST 1978

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

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Dam Safety

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

This report cites results of a technical investigation as to the dam's adequacy. The inspection and evaluation of the dam is as prescribed by the National Dam Inspection Act, Public Law 92-367. The technical investigation includes visual inspection, review of available design and construction records, and preliminary structural and hydraulic and hydrologic calculations, as applicable. An assessment of the dam's general condition is included in the report.

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DEPARTMENT OF THE ARMY
PHILADELPHIA DISTRICT, CORPS OF ENGINEERS
CUSTOM HOUSE—2 D & CHESTNUT STREETS
PHILADELPHIA, PENNSYLVANIA 19106

IN REPLY REFER TO
NAPEN-D

Honorable Brendan T. Byrne
Governor of New Jersey
Trenton, New Jersey 08621

20 SEP 1978

Dear Governor Byrne:

Inclosed is the Phase I Inspection Report for Split Rock Pond Dam in Morris County, New Jersey which has been prepared under authorization of the Dam Inspection Act, Public Law 92-367. A brief assessment of the dam's condition is given on the first two pages of the report.

Based on visual inspection, available records, calculations and past operational performance, Split Rock Pond Dam, a high hazard potential structure, is judged to be in fair overall condition. The dam's spillway is considered adequate since the Probable Maximum Flood (PMF) would not overtop the dam. To insure adequacy of the structure, the following actions, as a minimum are recommended:

a. Within one month from the date of approval of this report, engineering studies and analysis should be initiated by a qualified professional consultant, engaged by the owner, to determine the dam's structural stability. Any remedial measures found necessary should be initiated within calendar year 1979.

b. Within one year from the date of approval of this report, the following actions should be taken:

(1) Vegetation and other materials in the road subgrade weep holes along the crest of the dam should be removed.

(2) An emergency operations procedure should be developed and made known to all operations personnel. This should include instruction of dam operations during emergencies, evacuation notification for downstream areas, and plans for emergency drawdown of the reservoir.

(3) If trees or other debris collect at the spillway inlet during or after severe storms, maintenance personnel should remove such trees or debris as soon as practicable in order to maintain maximum spillway capacity.

(4) The seepage from the abutment rock near the junctions with the downstream face of the dam is not considered detrimental to dam stability at the present time. The abutment seepage areas should be visually monitored in the future by the owner's personnel or others who make future inspections of the dam.

A copy of the report is being furnished to Mr. Dirk C. Hofman, New Jersey Department of Environmental Protection, the designated State Office contact for this program. Within five days of the date of this letter, a copy will also be sent to Congresswoman Helen S. Meyner of the Thirteenth District. Under the provisions of the Freedom of Information Act, the inspection report will be subject to release by this office, upon request, five days after the date of this letter.

Additional copies of this report may be obtained from the National Technical Information Services (NTIS), Springfield, Virginia, 22161 at a reasonable cost. Please allow four to six weeks from the date of this letter for NTIS to have copies of the report available.

An important aspect of the Dam Safety Program will be the implementation of the recommendations made as a result of the inspection. We accordingly request that we be advised of proposed actions taken by the State to implement our recommendations.

Sincerely yours,

Joel T. Callahan

1 Incl
As stated

JOEL T. CALLAHAN
Lieutenant Colonel, Corps of Engineers
Acting District Engineer

Cy furn:
Mr. Dirk C. Hofman, P.E., Deputy Director
Division of Water Resources
N. J. Dept. of Environmental Protection
P.O. Box 2809
Trenton, NJ 08625

SPLIT ROCK POND DAM (NJ00264)

CORPS OF ENGINEERS ASSESSMENT OF GENERAL CONDITIONS

This dam was inspected on 15 and 16 June 1978 by Michael Baker, Jr., Inc. Consulting Engineers under contract to the U. S. Army Engineer District, Philadelphia, in accordance with the National Dam Inspection Act, Public Law 92-367.

The Split Rock Pond Dam, a high hazard potential structure, is judged to be in fair overall condition. The dam's spillway is considered adequate since the Probable Maximum Flood (PMF) would not overtop the dam. To insure adequacy of the structure, the following actions, as a minimum are recommended:

a. Within one month from the date of approval of this report, engineering studies and analysis should be initiated by a qualified professional consultant, engaged by the owner, to determine the dam's structural stability. Any remedial measures found necessary should be initiated within calendar year 1979.

b. Within one year from the date of approval of this report, the following actions should be taken:

(1) Vegetation and other materials in the road subgrade weep holes along the crest of the dam should be removed.

(2) An emergency operations procedure should be developed and made known to all operations personnel. This should include instruction of dam operations during emergencies, evacuation notification for downstream areas, and plans for emergency drawdown of the reservoir.

(3) If trees or other debris collect at the spillway inlet during or after severe storms, maintenance personnel should remove such trees or debris as soon as practicable in order to maintain maximum spillway capacity.

(4) The seepage from the abutment rock near the junctions with the downstream face of the dam is not considered detrimental to dam stability at the present time. The abutment seepage areas should be visually monitored in the future by the owner's personnel or others who make future inspections of the dam.

APPROVED: Joel T. Callahan

JOEL T. CALLAHAN
Lieutenant Colonel, Corps of Engineers
Acting District Engineer

DATE: 20 September 1978

PHASE I REPORT
NATIONAL DAM SAFETY PROGRAM

Name of Dam - Split Rock Pond Dam, Morris County, New Jersey
Stream - Beaver Brook
Dates of Inspection - 15 and 16 June 1978

ASSESSMENT OF GENERAL CONDITIONS

Split Rock Pond Dam is a concrete gravity dam with a maximum height of 39 feet and a crest length of 490 feet. The dam is owned and operated for standby water supply by the City of Jersey City.

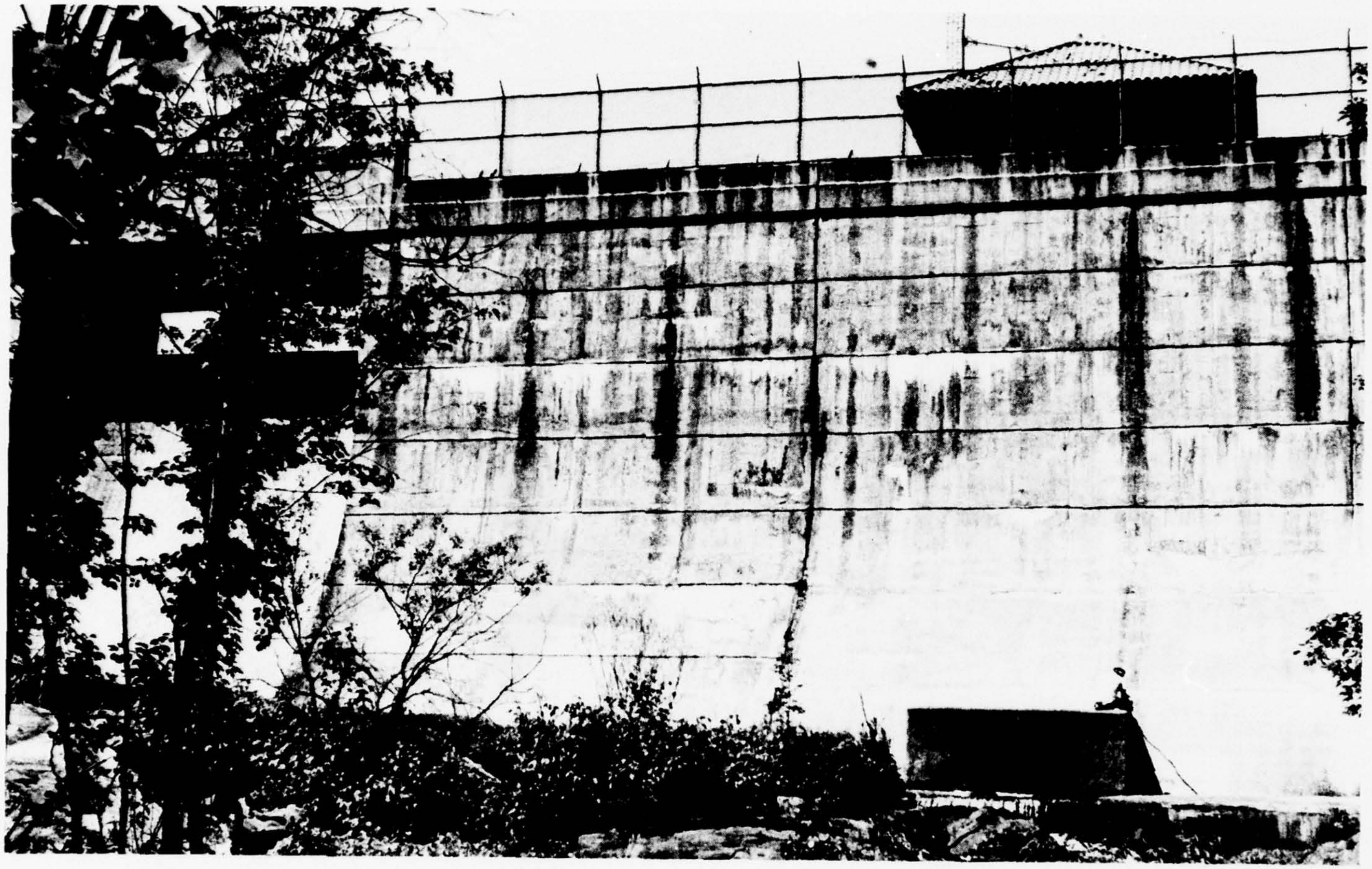
Visual inspections and review of engineering data in June 1978 indicate no serious deficiencies requiring emergency attention. The dam was found to be in fair overall condition at the time of inspection. The inspection disclosed potential problems with concrete deterioration and leakage of water from the downstream face of the dam. The owner should immediately retain a qualified consultant for further investigations of structural aspects of the dam, including structural stability. It is further recommended that the owner remove vegetation and other materials from road subgrade weep holes along the dam crest; develop emergency operations procedures for the dam and reservoir, including emergency warning and evacuation plans for areas which will be affected in the event of a dam failure; and remove trees or other debris from the spillway inlet as expeditiously as possible if such trees or debris collect at the spillway inlet during or after severe storms. The abutment seepage areas observed during the visual inspection should be visually monitored in the future by the owner's personnel or others who make future inspections of the dam.

Hydraulic/hydrologic evaluations performed in accordance with established Corps of Engineers procedures for Phase I Inspection Reports revealed that the spillway is considered adequate to pass the Probable Maximum Flood without overtopping the dam.

MICHAEL BAKER, JR., INC.

Michael Baker, III, P.E.
Chairman of the Board and
Chief Executive Officer
Registration Number 13385

NAME OF DAM: SPLIT ROCK POND DAM



OVERALL VIEW OF DAM

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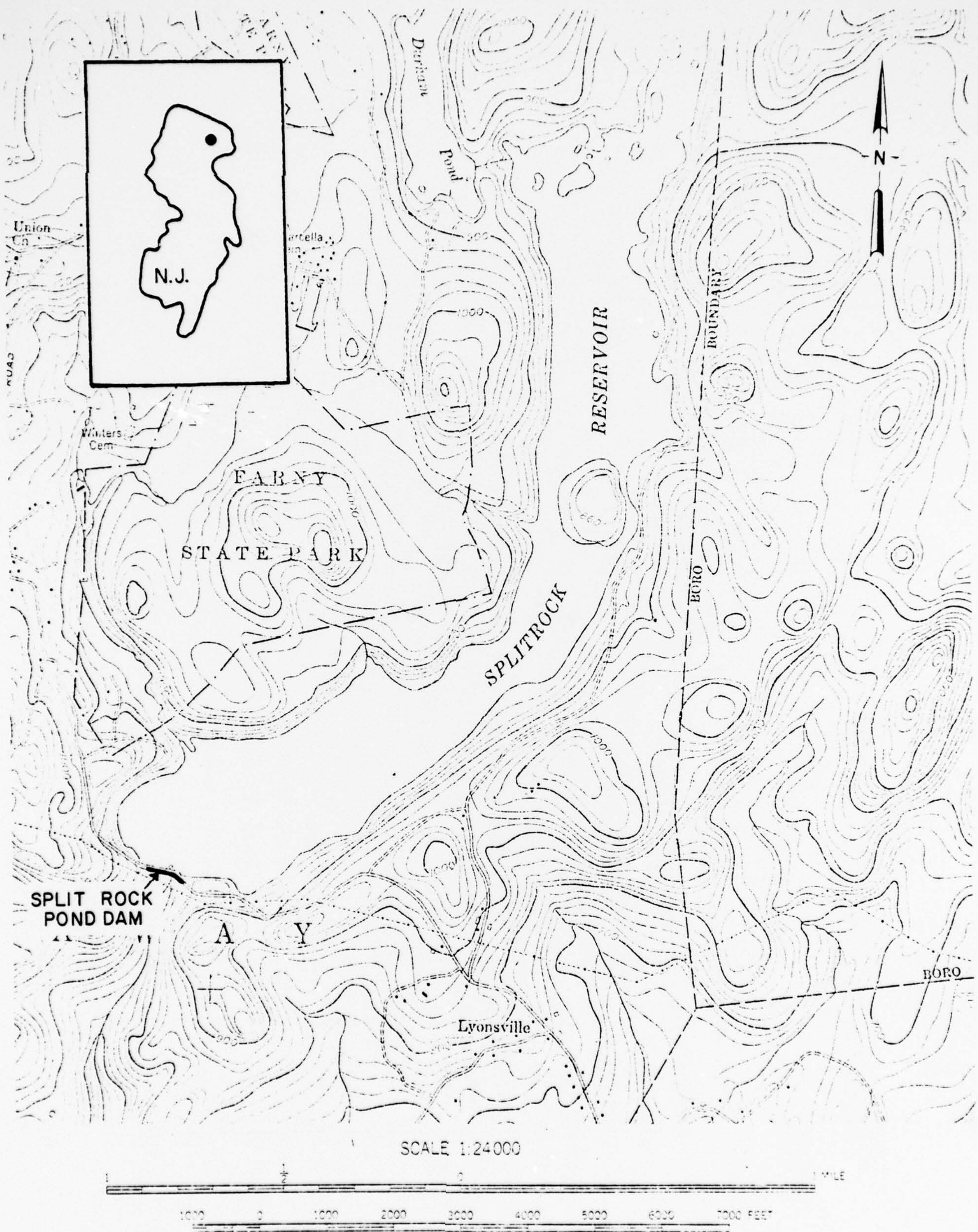
NAME OF DAM: SPLIT ROCK POND DAM



LOCATION PLAN
SPLIT ROCK POND DAM

1

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM
NAME OF DAM: SPLIT ROCK POND DAM, ID# NJ 00264

SECTION 1 - PROJECT INFORMATION

1.1 GENERAL

a. Authority - This report is authorized by the National Dam Inspection Act, Public Law 92-367, 92nd Congress, H.R. 15951 enacted 8 August 1972 and has been prepared in accordance with Contract No. DACW61-78-C-0141 between Michael Baker, Jr., Inc., and the U.S. Army Corps of Engineers, Philadelphia District.

b. Purpose of Inspection - The purpose of this inspection is to evaluate the general condition of Split Rock Pond Dam with respect to safety of the facility based upon available data and visual inspection.

1.2 DESCRIPTION OF PROJECT

a. Description of Dam and Appurtenances - Split Rock Pond Dam is a concrete gravity dam of cyclopean masonry. The dam has a maximum height of 39 feet, a crest length of 490 feet, and a crest width of 18 feet. Split Rock Road extends along the crest of the dam on a 14 feet wide roadway and a two span bridge over the spillway (described below). Roadway approach sections extend from the gravity dam into each abutment. The approach section on the left (east) abutment is approximately 160 feet long and the approach section on the right (west) abutment is approximately 290 feet long. Both approach sections have minor cuts and fills with heights of less than 10 feet. Concrete gravity walls with heights of less than 12 feet retain fill on the upstream sides of the two approach sections which lie almost entirely above normal pool level (spillway crest El. 835.0).

The dam is founded on gneiss bedrock, portions of which were grouted during the dam construction in 1948. No information is presently available on the type or extent of foundation or abutment grouting. The dam has no foundation drains and no seepage control measures other than the above mentioned foundation grouting.

The dam and reservoir provide standby water supply for the City of Jersey City. When necessary, water is released to flow via Beaver Brook and the

Rockaway River to Jersey City's Boonton Reservoir which is located about six miles southeast of Split Rock Pond Dam. The intake structure for Split Rock Pond Dam is a reinforced concrete gate chamber located on the left side of the stream channel. Reservoir water enters the gate chamber through two 30 by 30 inch electrically operated sluice gates and is discharged through a 36 inch diameter cast-iron outlet pipe located in the base of the concrete gravity section.

Split Rock Pond Dam has an ungated concrete ogee spillway located on the right side of the stream channel. The spillway has two 25 feet wide overflow sections separated by a three feet wide pier. Split Rock Road passes over the spillway on a two span reinforced concrete bridge. The spillway crest is at El. 835.0 and the base of the spillway bridge is at El. 840.0. The spillway discharges onto concrete slabs which step up the right abutment from approximately El. 810 at the left side of the spillway to approximately El. 820 at the right side of the spillway. The stream channel downstream from these concrete slabs consists of steeply sloping gneiss bedrock which is jointed, hard and essentially unweathered.

b. Location - Split Rock Pond Dam is located on Beaver Brook in Rockaway Township, Morris County, New Jersey. The dam is located about five miles north of Interstate Route 80 and U.S. Route 46 at the town of Denville and about five miles northwest of Interstate Route 287 at the town of Boonton. Access to the dam is provided by Split Rock Road which extends along the crest of the dam and crosses the spillway on a bridge.

c. Size Classification - The maximum height of the dam is 39 feet and the reservoir volume to the top of the dam is 13,874 acre-feet. Therefore, the dam is in the "Intermediate" size category as defined by the "Recommended Guidelines for Safety Inspection of Dams."

d. Hazard Classification - The reach of Beaver Brook which extends approximately one mile downstream from the dam passes through an uninhabited narrow valley to the village of Meriden. The next reach of Beaver Brook extends approximately two miles further downstream from Meriden through a wider, swampy valley to the village of Beach Glen. The portion of the valley from Meriden to Beach Glen is sparsely populated with an estimated 20 homes and 100 people situated in areas which might be affected by flooding. Additional homes and other structures and facilities are located further downstream along Beaver Brook from Beach Glen to the Rockaway River. In the event of failure of Split Rock Pond Dam, it is likely that "more than a few" lives would be lost and economic losses would be "appreciable" to "excessive." The dam is therefore considered to be in the "High" hazard category as defined in the "Recommended Guidelines for Safety Inspection of Dams."

e. Ownership - The dam is owned by Jersey City, New Jersey. The mailing address is: Division of Water, Jersey City Public Works Department, 575 Route 440, Jersey City, New Jersey 07305.

f. Purpose of Dam - The dam is used for water supply on a standby basis.

g. Design and Construction History - The dam was designed for the owner by Clyde Potts, Consulting Engineer, New York, New York in 1923. The design was revised slightly in 1946, and the dam was constructed in 1947 and 1948. Very little information is presently available on design and construction. According to George Plastoris, Jersey City Watershed Superintendent, the dam was constructed by Webster Construction Company.

h. Normal Operational Procedures - Efforts are made to keep Split Rock Reservoir full to spillway crest El. 835.0 at all times. The owner is required to release one m.g.d. into Beaver Brook to maintain minimum downstream flows. The sluice gates are set to maintain this minimum daily discharge and additional water sometimes flows over the spillway. According to George Plastoris, Jersey City Watershed Superintendent, a discharge of 20 m.g.d. is the maximum flow which will not cause downstream flooding problems. Mr. Plastoris further stated that water was last released from Split Rock Reservoir for supplemental water supply in 1977 when the reservoir was drawn down to approximately El. 830.

Mr. Plastoris, who has his office at Boonton Dam about six miles southeast from Split Rock Pond Dam, personally adjusts the sluice gates at Split Rock Pond Dam when discharges in excess of the basic one m.g.d. are required. He maintains a close watch on the sluice gates, especially during periods of heavy rainfall. In addition, Split

Rock Pond Dam is visited daily by a water works employee who records water levels in the reservoir and at a stream gaging weir located about 180 feet downstream from the dam. Routine maintenance of the dam and appurtenant structures is done as necessary and/or when funds are available.

Additional operations and maintenance information may be available in the files of the Jersey City Public Works Department. The information summarized above, which was obtained from the owner's representatives during field inspection of the dam, are considered sufficient for purposes of this Phase I Inspection Report.

1.3 PERTINENT DATA

a. Drainage Area - The drainage area of Split Rock Pond Dam is 5.6 square miles.

b. Discharge at Damsite - The maximum known flow at the damsite is not available.

c. Elevation [feet above Mean Sea Level (M.S.L.)] -

Top of Dam - 843.5 (top of crest parapet, maximum structural height)
842± (low points for overtopping abutment approach roads)
840.0 (base of spillway bridge)
Maximum Pool (Design Discharge) - approximately 837.6
Normal Pool - 835.0
Streambed at Centerline of Dam - 805
Maximum Tailwater - Not available

d. Reservoir (miles) -

Length of Normal Pool - 3.27

e. Storage (acre-feet) -

At Spillway Crest (El. 835.0) - 9517
Top of Dam (El. 842) - 13,874

f. Reservoir Surface (acres) -

Top of Dam (El. 842) - 694
Spillway Crest - 566
Normal Pool - 566

g. Dam -

Type - Concrete gravity (cyclopean masonry)
Length - 490 feet
Height - 39 feet
Top Width - 18 feet
Side Slopes - Upstream - vertical
Downstream - variable
Impervious Core - Not applicable
Cutoff - Some grouting of gneiss foundation rock done in 1948

h. Diversion and Regulating Tunnel - None

i. Spillway -

Type - Concrete ogee
Length of Weir - 50 feet
Crest Elevation - 835.0 feet (M.S.L.)
Gates - None
Downstream Channel - Steeply sloping unweathered gneiss bedrock

j. Regulating Outlets - Two 30 inch by 30 inch electrically operated sluice gates leading to 36 inch diameter cast-iron outlet pipe with invert El. 802.0.

SECTION 2 - ENGINEERING DATA

2.1 DESIGN

Design information reviewed included:

1) Drawings - "Impounding Reservoir at Split Rock Pond for Jersey City, N.J." Fifteen sheets of plans, sections and details prepared by Clyde Potts, Consulting Sanitary Engineer, New York, New York, in September 1923 and revised slightly in May 1946. (A set of prints of these drawings was borrowed from the Jersey City Public Works Department (J.C.P.W.D.); the original drawings were reportedly lost several years ago.)

2) Notes, Correspondence, Calculations, and Memoranda in the microfiche files of the New Jersey Department of Environmental Protection (N.J.D.E.P.).

Other design information may exist in the files of Clyde Potts' successors if such successors indeed exist. Additional design information reportedly exists in the inactive files of the J.C.P.W.D. This latter information was requested; but, as of this writing (28 July 1978), no information has yet been received.

2.2 CONSTRUCTION

Very little information on the construction of Split Rock Pond Dam was available for review in connection with this Phase I Inspection Report. The limited information in the microfiche files of the N.J.D.E.P. indicates that dam construction began in late August 1947; no work was done over the winter from late December 1947 to late March 1948; foundation grouting was completed in July 1948; the dam was completed in the autumn of 1948; and it was dedicated on 19 November 1948. No "as built" drawings of the dam were available for review. Observations during the field inspection indicate that visible portions of the dam were generally constructed in accordance with Clyde Potts' May 1946 revised design drawings. However, details of foundation construction are unknown.

As indicated in paragraph 2.1 with regard to design information, additional information on construction of Split Rock Pond Dam reportedly exists in the inactive files of the J.C.P.W.D. This information was requested, but it has not yet been received.

2.3 OPERATION

Very little information on the operation of Split Rock Pond Dam and Reservoir was available for review in connection with this Phase I Inspection Report.

The information summarized in paragraph 1.2.h. of this report was obtained in an interview with George Plastoris, Watershed Superintendent, and other representatives of the J.C.P.W.D. More detailed operations information is probably available in their files.

2.4 EVALUATION

The readily available design, construction and operations information, plus observations made during the field inspection, are considered adequate for purposes of this Phase I Inspection Report on Split Rock Pond Dam.

SECTION 3 - VISUAL INSPECTION

3.1 FINDINGS

a. General - The dam and its appurtenant structures were found to be in fair overall condition at the time of inspection. Most of the problems noted during the visual inspection are considered minor and do not require immediate remedial treatment. The one problem requiring immediate attention is the general deterioration and cracking of concrete in the dam and the seepage of water through cracks and horizontal and vertical joints in the downstream face of the dam. It is impossible to assess from the type of visual inspection performed for this Phase I Inspection Report the extent to which the concrete deterioration and cracking and the observed seepage will affect the structural integrity of the dam. For this reason, it is strongly recommended that the J.C.P.W.D. immediately retain a consultant experienced in stability evaluations of concrete gravity dams to:

1) Evaluate structural aspects, including structural stability, of the dam as it presently exists.

2) Develop plans and specifications for remedial work as necessary.

Significant observations made during visual inspection of Split Rock Pond Dam by Michael Baker, Jr., Inc. are presented briefly in the following paragraphs. The complete visual inspection check list is given in Appendix A.

b. Dam - Considerable deterioration, i.e., cracking, spalling, leaching and mineral precipitation, was observed in most of the exposed concrete surfaces of the dam.

An area of bulged and cracked concrete approximately three feet high by ten feet wide was observed on the left side of the downstream face of the dam. The concrete appeared to be bulged outward approximately one-half an inch in this area.

Minor leakage of water (presumably from the reservoir) was observed from several horizontal construction joints, vertical monolith joints, and miscellaneous cracks in the downstream face of the dam. None of these leaks was flowing at a perceptible rate and all leakage evaporated when the sun reached the downstream face of the dam.

Many of the six inch diameter weep holes for drainage of the subgrade of the road along the dam crest were partially or fully plugged with various vegetation and other material. A six inch diameter tree was growing from a weep hole on the left side of the downstream face of the dam. Smaller trees and shrubs were growing from other weep holes on the upstream and downstream faces of the dam.

Minor seepage was observed from the jointed gneiss bedrock in both abutments near the junctions with the downstream face of the dam. The top of the seepage was at approximately El. 800 on the left abutment and at approximately El. 820 on the right abutment. Both of these seepage areas had flows estimated at approximately one g.p.m. and appearances suggestive of long term steady seepage through rock joints.

3.2 EVALUATION

a. Dam - As indicated in paragraph 3.1.a., it is impossible to assess, from the results of a visual inspection of this type, the extent to which concrete cracking, bulging, deterioration and leakage may affect the structural integrity of the dam. The nature and potential implications of these phenomena are such that the owner should immediately retain a properly qualified consultant to make more detailed investigations of structural aspects, including structural stability, of the dam as it presently exists and to develop plans and specifications for remedial work as necessary.

Vegetation and other materials in the road subgrade weep holes along the crest of the dam are not considered detrimental to dam stability at the present time. The weep holes should be cleaned out and vegetation should be removed in the course of routine maintenance. The consultant who investigates structural aspects of the dam, as recommended above, may provide additional guidance relative to weep hole maintenance.

Seepage observed from the abutment rock near the junctions with the downstream face of the dam is not considered detrimental to dam stability at the present time. The abutment seepage areas should be visually monitored by the owner's personnel or others who make future inspections of the dam.

SECTION 4 - OPERATIONAL PROCEDURES

4.1 PROCEDURES

Operational procedures are generally discussed in paragraph 1.2.h.

There is no formal written procedure for reservoir operation or for emergency downstream evacuation in the event of impending catastrophe. However, operating personnel interviewed have a keen sense of awareness and access to good telephone communication with which to alert civil defense and police.

Rapid emergency drawdown is impossible with the 36 inch pipe which is the only reservoir outlet when the pool is below El. 835, spillway crest level.

It is recommended that a formal emergency procedure be prepared and prominently displayed and furnished to all operating personnel. This should include:

1) How to operate the dam and reservoir during an emergency.

2) Procedures for evaluating inflow and for rapid drawdown of the reservoir during periods of emergency operation.

3) Who to notify, including public officials, in case evacuation from the downstream area is necessary.

In addition, the owner should assist public officials in developing an emergency evacuation plan for areas which will be affected in the event of a dam failure.

4.2 MAINTENANCE OF DAM

Maintenance of the dam should be improved with regard to the concrete surfaces and road subgrade weep holes.

4.3 MAINTENANCE OF OPERATING FACILITIES

Maintenance of the operating facilities is generally adequate.

4.4 WARNING SYSTEMS

There are no warning systems or special emergency procedures in the event of a dam failure. However, the local civil defense and police officials would be notified. An emergency warning and evacuation plan should be developed as recommended in paragraph 4.1 above.

4.5 EVALUATION

Operating procedures are adequate for their intended purposes. Certain aspects of maintenance should be improved as noted in paragraph 4.2. A formal emergency procedure should be developed as recommended in paragraph 4.1.

SECTION 5 - HYDRAULIC/HYDROLOGIC

5.1 EVALUATION OF FEATURES

a. Design Data - Some hydraulic and hydrologic design information was available in the microfiche files of the N.J.D.E.P. This information included a summary of design data provided by Clyde Potts, the dam designer, as well as more detailed calculations made by engineers of the New Jersey Department of Conservation during review of the dam permit application in 1947. Our review of this information indicates that considerable attention was directed toward ensuring the hydraulic safety of the dam. According to a summary statement in the microfiche files, the spillway was found to have a capacity of nearly two times the discharge from a rainfall of 200 year recurrence interval, based on Yarnell's Curve.

b. Experience Data - Only minimal experience data were available for review and evaluation. George Plastoris, Jersey City Watershed Superintendent, indicated that the maximum pool of record was at El. 836.50; but we do not presently know the date of this maximum pool or the reservoir level prior to the precipitation event associated with this maximum pool. Our calculations indicate a spillway discharge of approximately 326 c.f.s. with this maximum pool of record.

c. Visual Observations - Observations during field inspection of the dam indicate that the spillway functions in the manner for which it was designed. The sluice gates which are used for the constant discharge of one m.g.d. as well as for drawdown of the reservoir appeared to be operating properly. Mr. Plastoris stated that the operation of the gates is checked on an annual basis.

d. Overtopping Potential - Split Rock Dam is classified as a "High" hazard-"Intermediate" size dam requiring evaluation for a Spillway Design Flood (S.D.F.) equal to the Probable Maximum Flood (P.M.F.). The spillway consists of two 25 feet long concrete ogee weirs, both having a crest El. 835 feet M.S.L. The bridge constructed over the spillway has base El. 840 feet M.S.L. The lowest points on the bridge deck and on the abutment area approach roads have El. 842 feet M.S.L. which corresponds to the minimum top of dam elevation. Spillway rating curves were developed by computing weir and orifice flow as presented in Design of Small Dams by the U.S. Bureau of Reclamation and American Civil Engineers Handbook, fifth edition, by Merriman and Wiggin, 1942, John Wiley & Sons Inc., respectively. The rating curve shows a maximum weir discharge of about 2279 c.f.s. at El. 840 and a maximum discharge of about 2724 c.f.s. at top of dam El. 842. Split Rock Pond Dam also has a gate house which regulates a sluice-gated outlet used for reservoir drawdown. The discharge through this outlet was not considered in the hydraulic computations since the outlet, which must be operated manually, cannot be assumed to be open at the time the peak inflow occurs.

The hydrologic analysis was completed by the use of the Flood Hydrograph Computer Package HEC-1 prepared by the U.S. Army Corps of Engineers in conjunction with procedures outlined in the Design of Small Dams by the U.S. Bureau of Reclamation. The rainfall depth was obtained from Technical Paper 40 by the U.S. Weather Bureau. Design plans and field measurements were used for spillway dimensions.

Using the flood routing option of HEC-1 and the above data, the S.D.F. (equal to the P.M.F.) was routed through the reservoir. The results indicated that the spillway capacity was adequate to pass the P.M.F. without overtopping the dam. The reservoir would reach a maximum level of El. 840.5 feet M.S.L. while passing the P.M.F.

The conclusions presented in this Phase I Inspection Report pertain to present conditions, and the effect of future development on the hydrology has not been considered.

e. Emergency Drawdown - Drawdown of the reservoir can be accomplished by opening the sluice gates so that water is released through the 36 inch cast-iron outlet pipe with invert El. 802.0. With a maximum discharge of 200 c.f.s. at normal pool, the reservoir can be drained from the spillway crest level in approximately 48 days.

SECTION 6 - STRUCTURAL STABILITY

6.1 EVALUATION OF STRUCTURAL STABILITY

a. Visual Observations - No structural inadequacies were noted during visual inspection of the dam; but considerable concrete deterioration, i.e., cracking, spalling, leaching and mineral precipitation, was observed along with leakage of minor amounts of water from horizontal and vertical joints and various cracks in the downstream face of the dam. It is impossible to assess, from the results of a visual inspection of this type, the extent to which the observed features may affect the structural integrity of the dam. As noted in Section 3 of this report, the nature and potential implications of these phenomena are such that the owner should immediately retain a consultant experienced in stability evaluations of concrete gravity dams to:

1) Evaluate structural aspects, including structural stability, of the dam as it presently exists.

2) Develop plans and specifications for remedial work as necessary.

b. Design and Construction Data - The microfiche files of the N.J.D.E.P. contain a foundation sliding stability analysis performed by N. C. Wittwer of the New Jersey Department of Conservation on 3 March 1947 in connection with review of the dam permit application. This analysis was based on uplift and other assumptions which, while reflecting the state of the art 30 years ago, are of doubtful validity in terms of modern criteria. Results of that stability analysis are therefore considered inconclusive.

As indicated in Section 2 of this report, "as built" foundation information is not readily available on the dam. Experience with analyses of other gravity dams indicates that Split Rock Pond Dam could probably be shown to satisfy the foundation sliding stability requirements outlined in Section 4.4.4.5 of the "Recommended Guidelines for Safety Inspection of Dams." This should be verified, however, by the consultant who evaluates structural aspects, including stability, of the dam as recommended in paragraph 6.1.a.

c. Operating Records - Nothing in the readily available operating records suggests structural inadequacy of the dam.

d. Post-Construction Changes - Aside from the concrete deterioration and leakage mentioned previously, there are no post-construction changes which affect structural adequacy of the dam.

e. Seismic Stability - Split Rock Pond Dam is located in Seismic Zone 1 according to the "Seismic Zone Map of the Contiguous United States" Figure 1, page D-30, "Recommended Guidelines for Safety Inspection of Dams." This is a zone of low seismic activity. Experience indicates that dams in Seismic Zone 1 will have adequate stability under seismic loading conditions, if they have adequate stability under static loading conditions. This point should be addressed further by the consultant who evaluates structural aspects, including stability, of the dam as recommended in paragraph 6.1.a.

SECTION 7 - ASSESSMENT, RECOMMENDATIONS/REMEDIAL MEASURES

7.1 DAM ASSESSMENT

a. Safety - There are no findings, as a result of this inspection, from which a detrimental assessment can be rendered. No structural inadequacies were noted during visual inspection of the dam but considerable concrete deterioration along with leakage of water from the downstream face of the dam was observed, as described in Section 3 of this report. The nature and potential implications of these concrete deterioration and leakage phenomena are such that additional investigations of structural aspects, including structural stability, of the dam are considered necessary. More detailed recommendations in this regard are given below in paragraph 7.2.

The spillway capacity was analyzed using the criteria presented in the "Recommended Guidelines for Safety Inspection of Dams" and according to the procedures presented in paragraph 5.1.d. The analysis determined that the spillway is considered adequate to pass the P.M.F. without overtopping the dam.

b. Adequacy of Information - The readily available information and the observations made during field inspection of the dam are considered sufficient for purposes of this Phase I Inspection Report.

c. Urgency - The owner should immediately retain a qualified consultant to perform additional investigations of structural aspects, including structural stability, of the dam. Results of these additional investigations will clarify the need for remedial work, if any, and the urgency of such remedial work.

7.2 RECOMMENDATIONS/REMEDIAL MEASURES

The dam inspection revealed one item requiring immediate attention. The owner should immediately retain a consultant experienced in stability evaluations for concrete gravity dams to:

1) Evaluate structural aspects, including structural stability, of the dam as it presently exists.

2) Develop plans and specifications for remedial work as necessary.

In connection with these investigations, the consultant should obtain and review any pertinent information on the design and construction of Split Rock Pond Dam which may exist in the files of the Jersey City Public Works Department. Stability evaluations of the dam should be done in accordance with procedures and criteria outlined in Section 4.4 of the "Recommended Guidelines for Safety Inspection of Dams." A copy of Section 4.4 is included in Appendix C of this report for reference purposes.

The dam inspection also disclosed several items of lower priority which should be performed in the near future. These are:

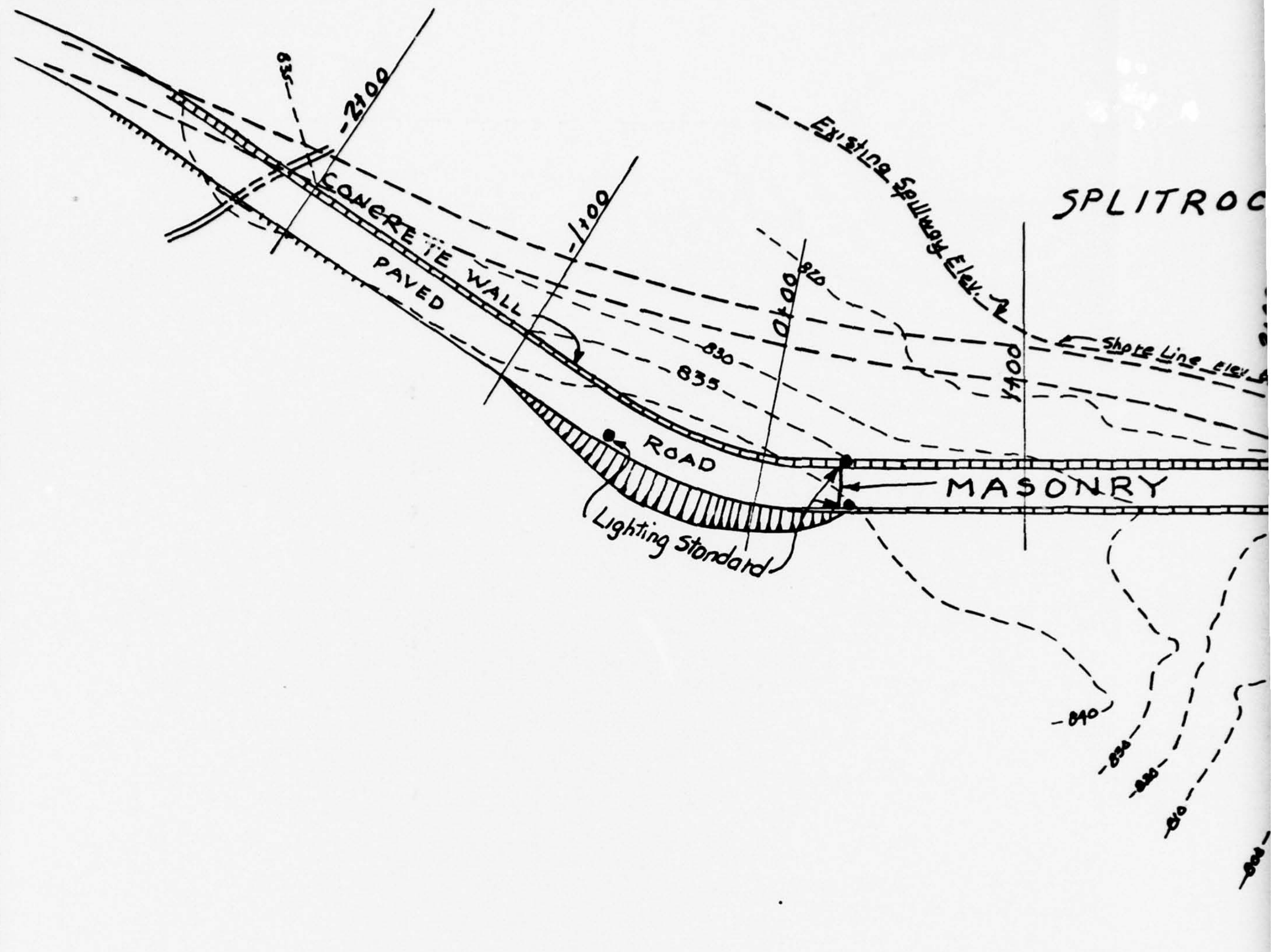
1) Vegetation and other materials in the road subgrade weep holes along the crest of the dam should be removed. (The consultant who investigates structural aspects of the dam may provide additional guidance relative to weep hole maintenance as well as maintenance of the dam in general.)

2) An emergency operations procedure should be developed and made known to all operations personnel. This should include instruction of dam operations during emergencies, evacuation notification for downstream areas, and plans for emergency drawdown of the reservoir.

3) If trees or other debris collect at the spillway inlet during or after severe storms, maintenance personnel should remove such trees or debris as soon as practicable in order to maintain maximum spillway capacity.

Further, the seepage from the abutment rock near the junctions with the downstream face of the dam is not considered detrimental to dam stability at the present time. The abutment seepage areas should be visually monitored in the future by the owner's personnel or others who make future inspections of the dam.

PLATES



Tracing of portion of original sheet no. 1 of 15 by MICHAEL BAKER, JR., INC. July 1978

SPLIT ROCK POND DAM
GENERAL PLAN

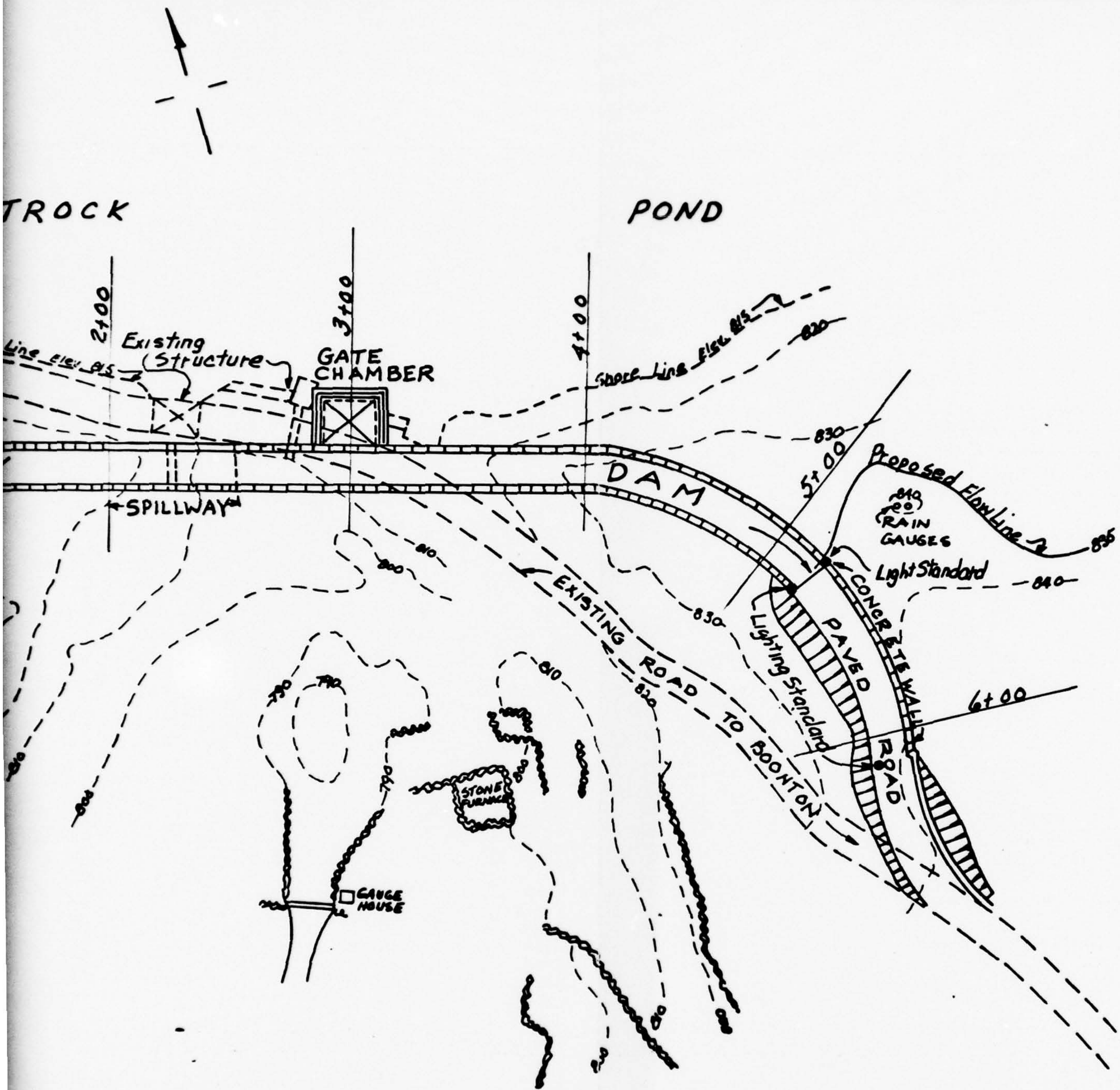
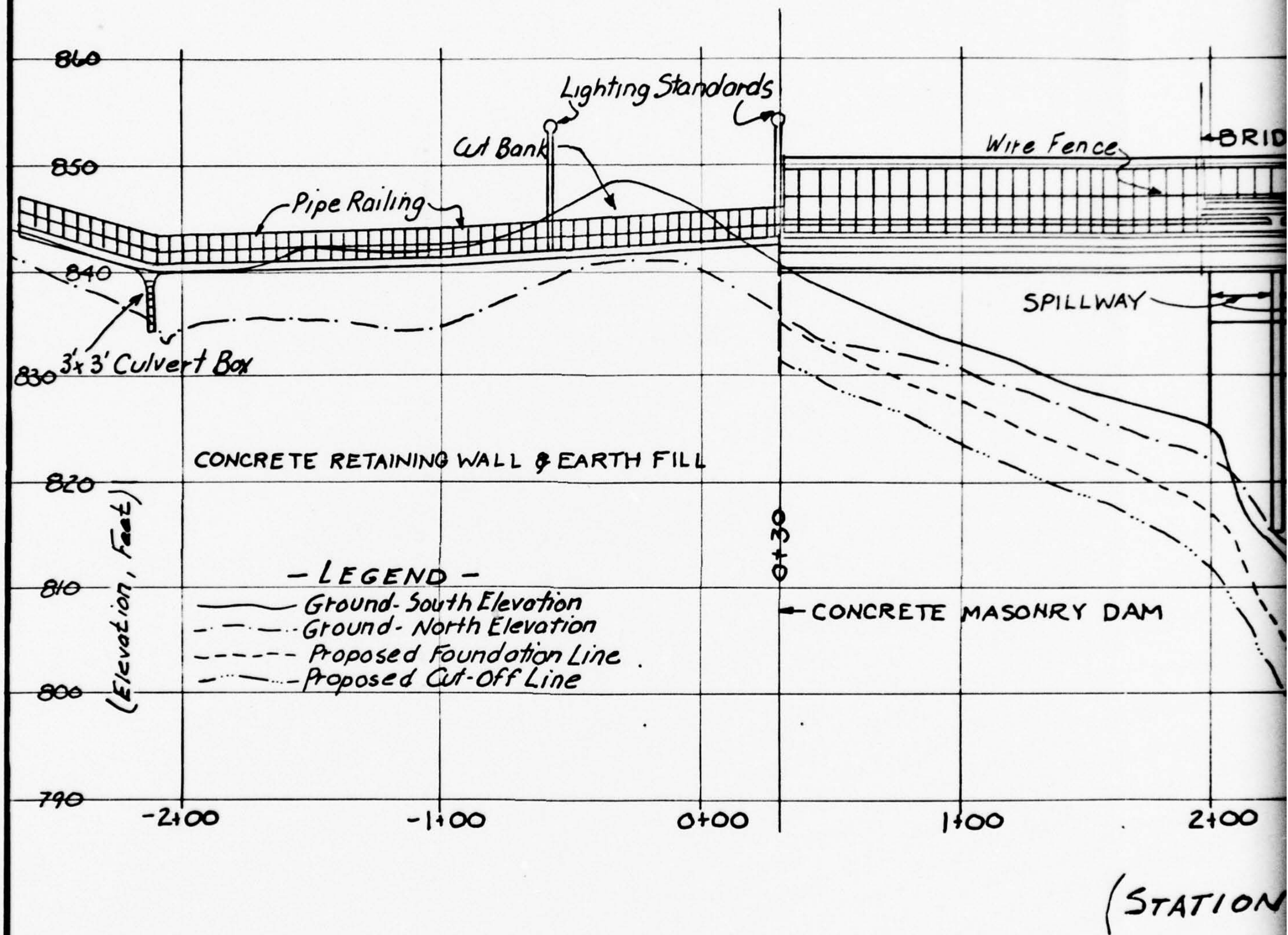


PLATE 1

60 40 20 0 20 40 60 Feet	DATE: MAY 1946

2



Tracing of portion of original sheet no. 1 of 15 by MICHAEL BAKER, JR., INC. July 1978

SPLIT ROCK POND DAM

DEVELOPED PROFILE - SOUTH ELEVATION (L

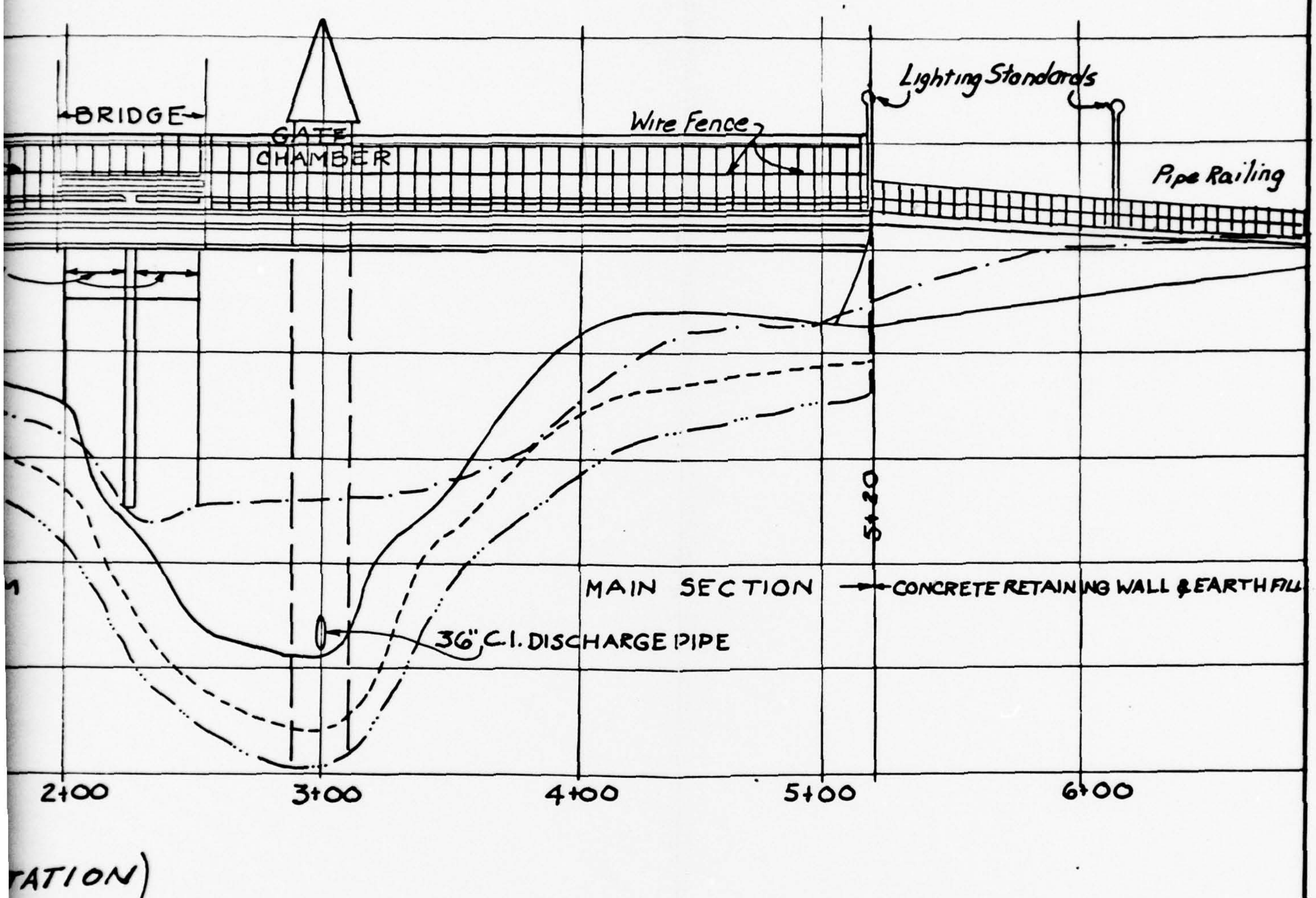


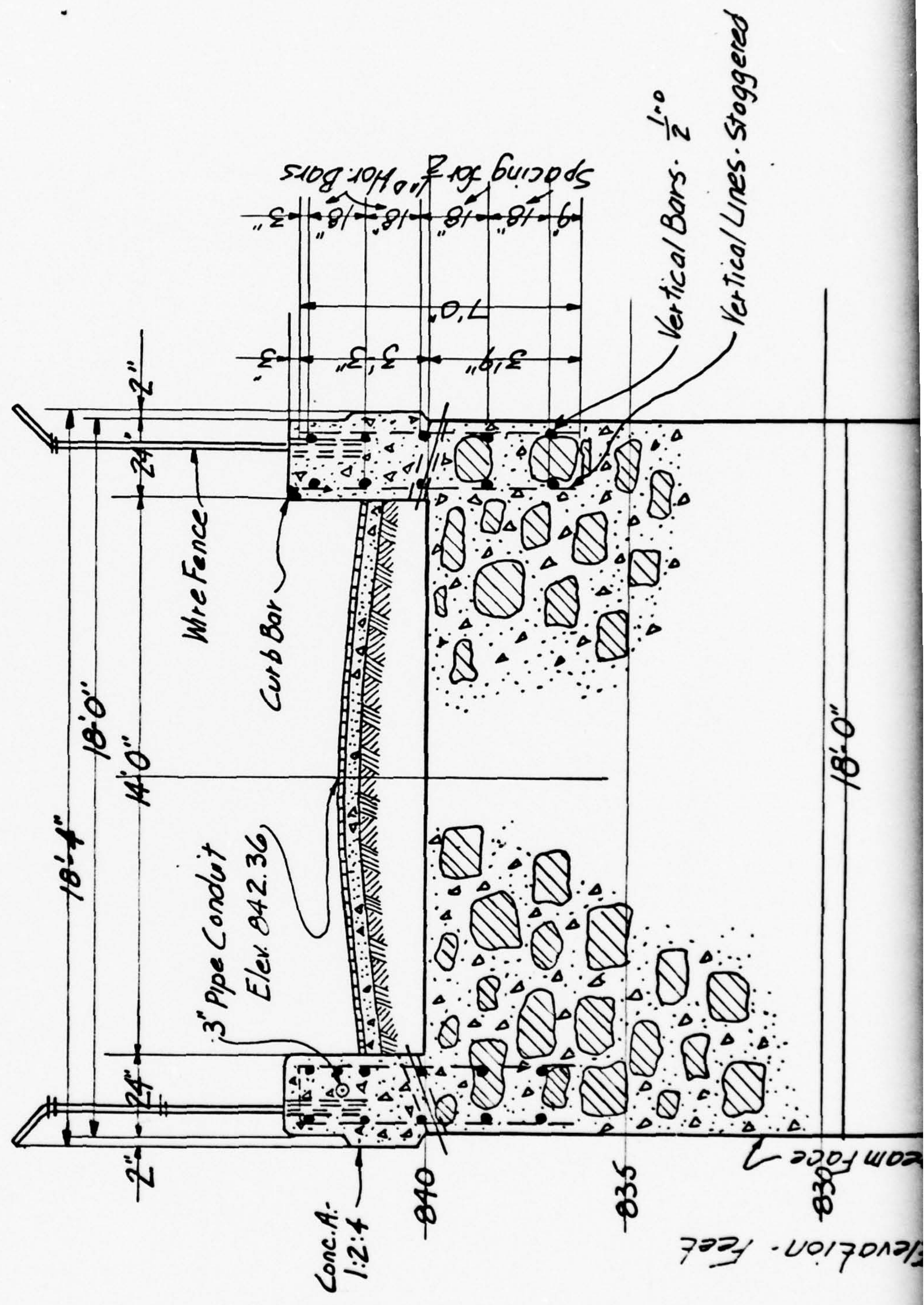
PLATE 2

TION (LOOKING UPSTREAM)

60 40 20 0 20 40 60
Horz. Feet

DATE: MAY 1946

2



Tracing of portion of original sheet no. 4 of 15 by MICHAEL BAKER, JR., INC. July 1978

SPLIT ROCK POND DAM
CROSS-SECTION STATION 3+46

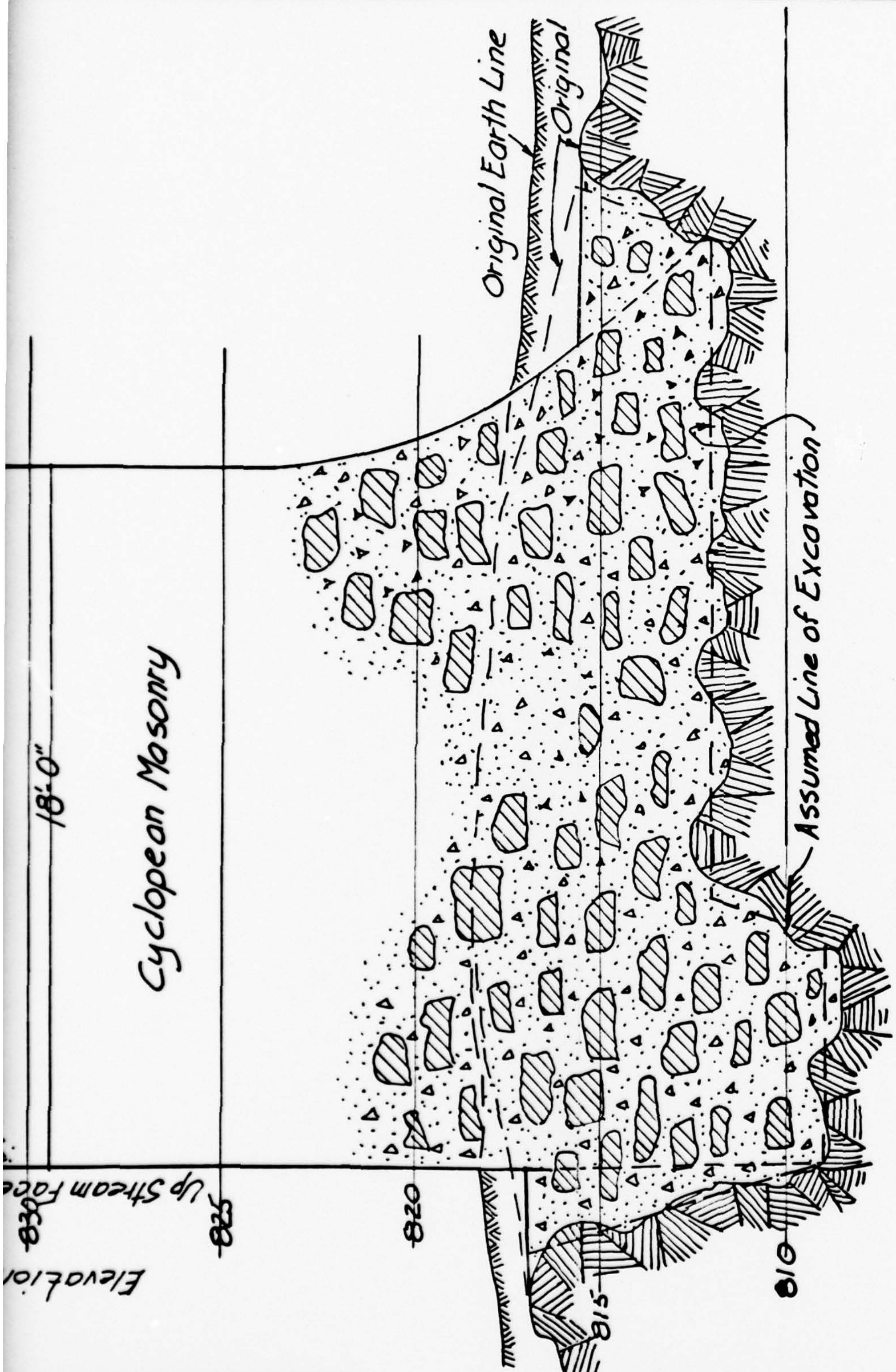
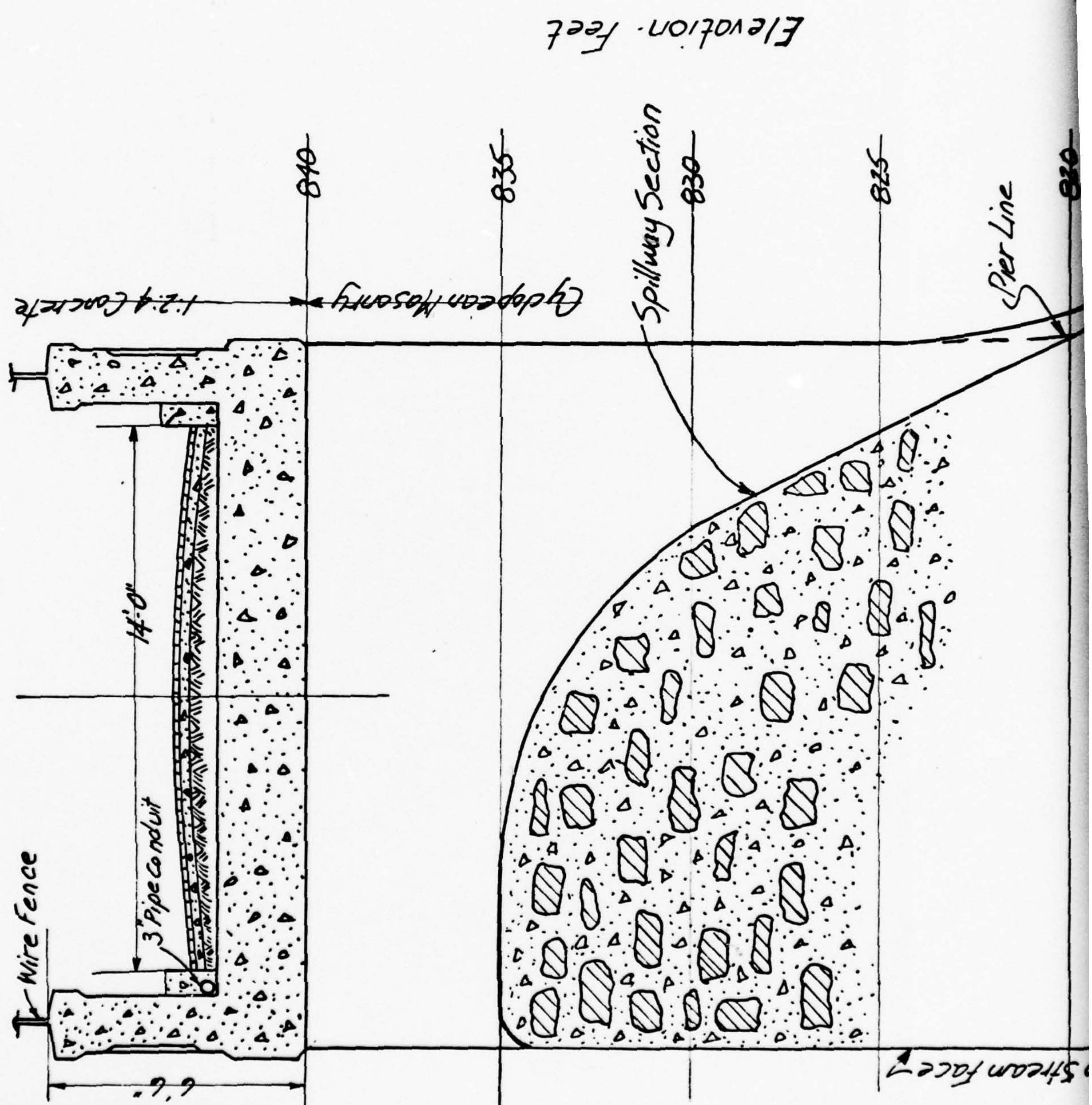


PLATE 3

40 30 20 10 0 20 40 Feet

DATE: MAY 1946



Tracing of portion of original sheet
no. 5 of 15 by
MICHAEL BAKER, JR., INC.
July 1978

SPLIT ROCK POND DAM
SPILLWAY CROSS-SECTION STATION 2+40

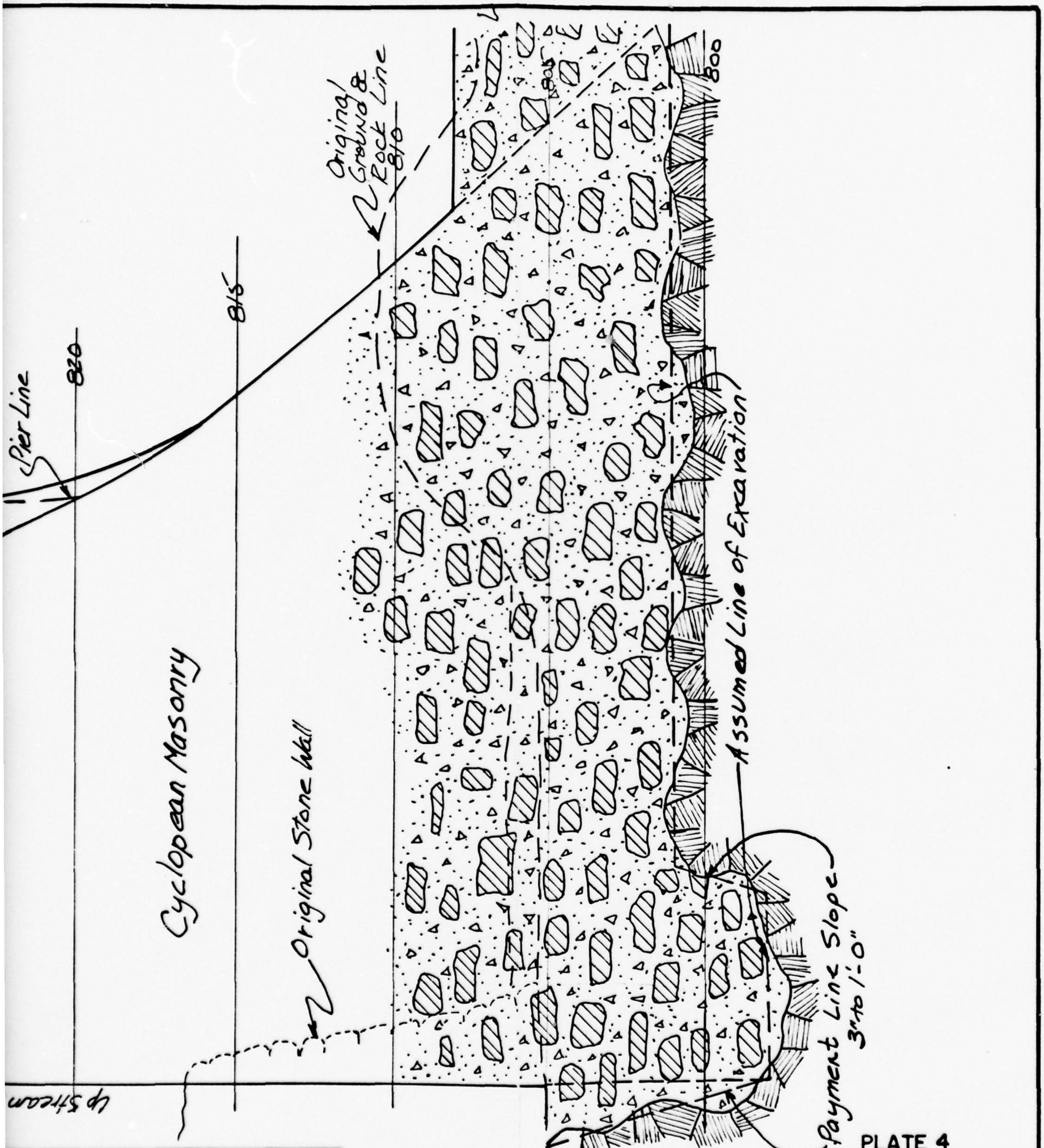
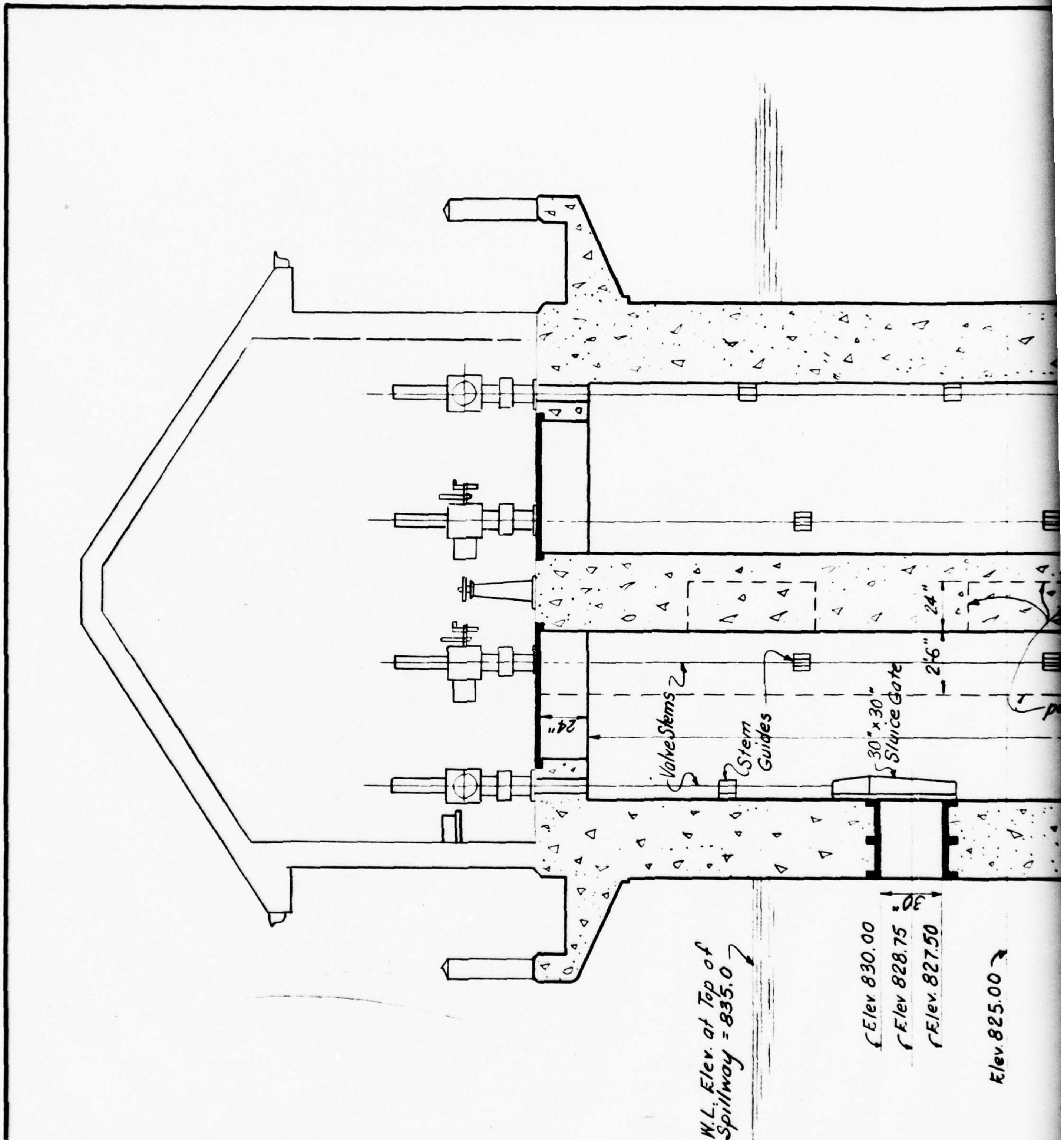


PLATE 4

2+40

40 30 20 10 0 20 40 Feet

DATE: MAY 1946



Tracing of portion of original sheet no. 8 of 15 by MICHAEL BAKER, JR., INC. Aug. 1978

SPLIT ROCK POND DAM

GATE CHAMBER PROFILE

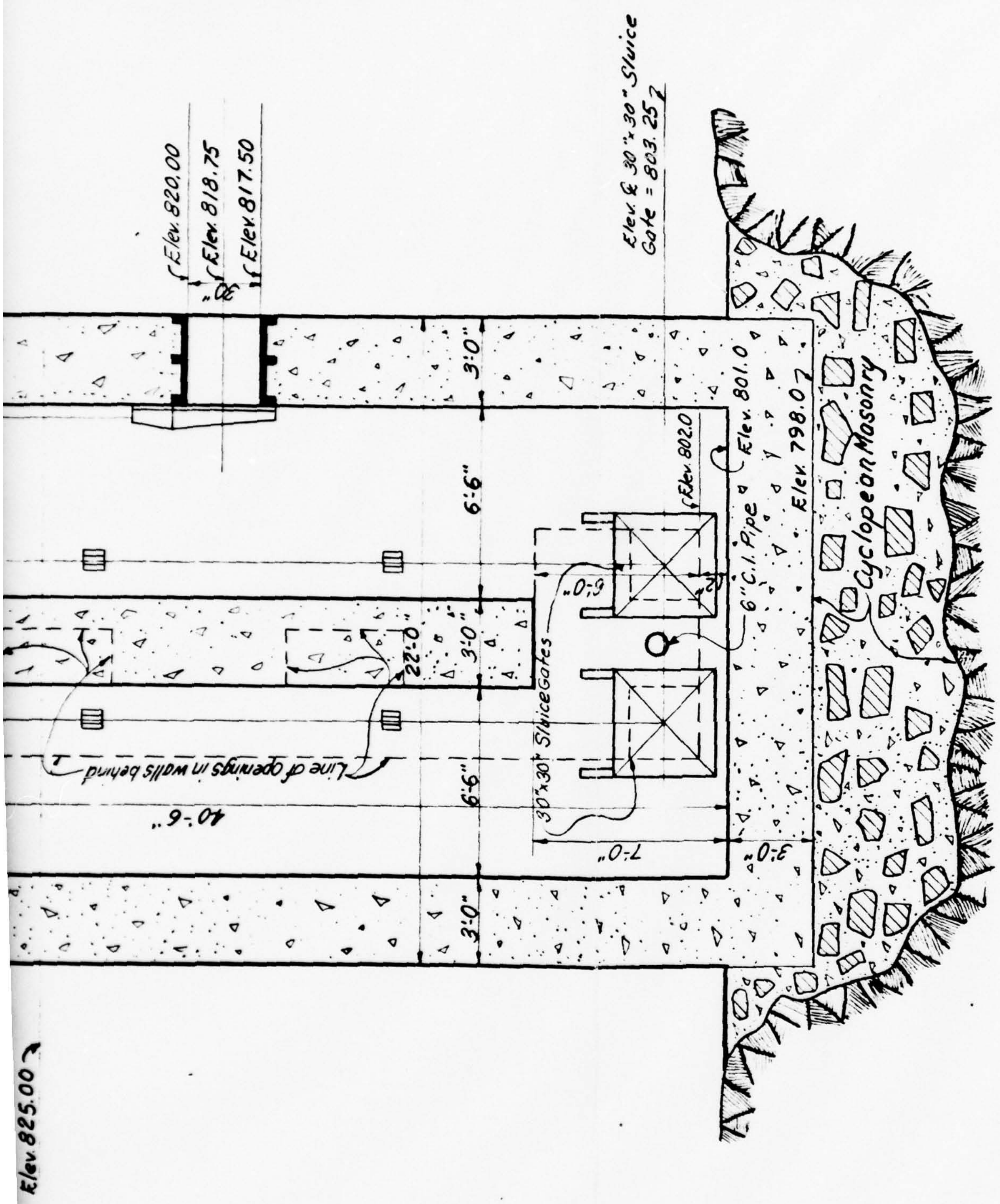


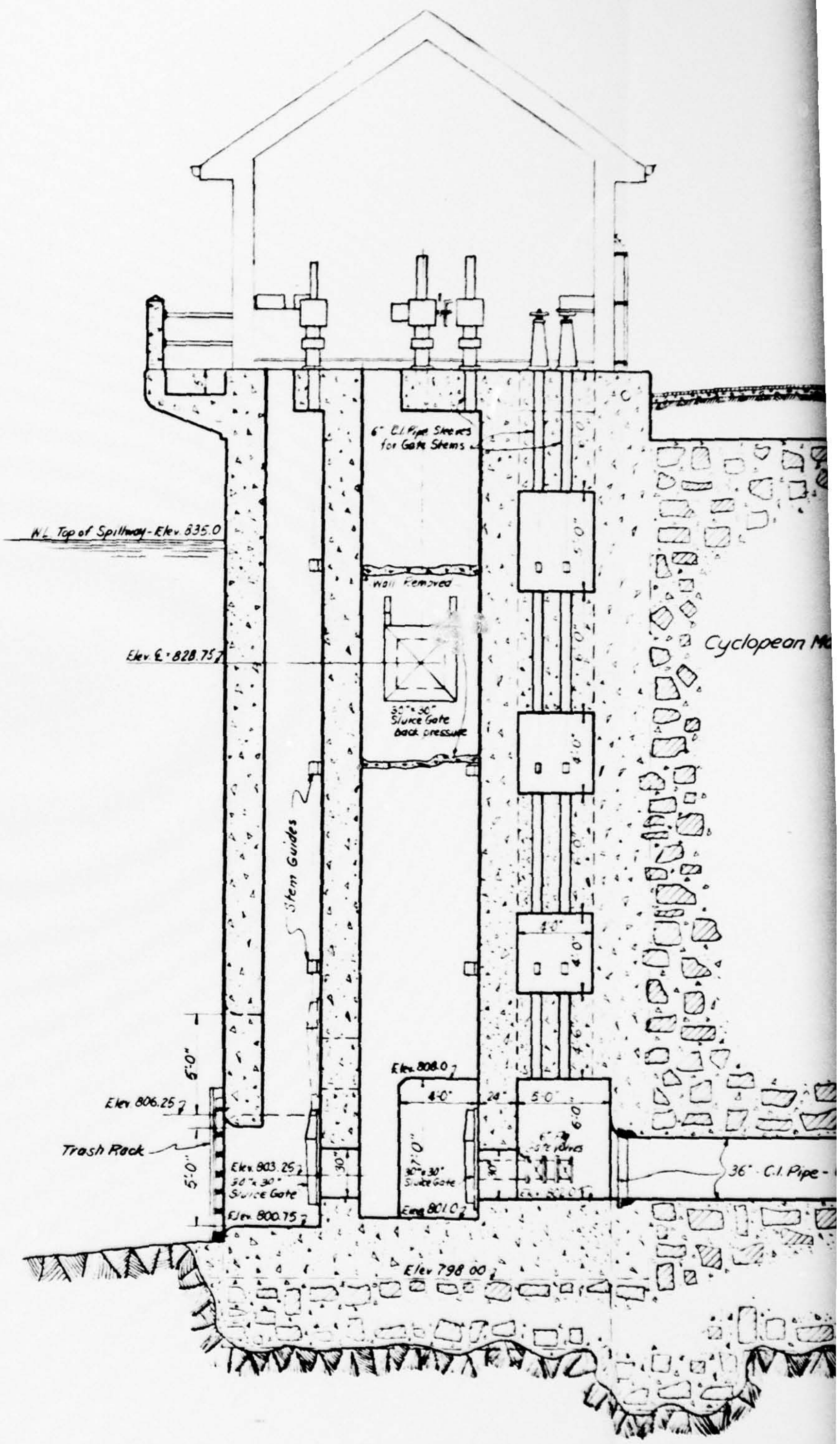
PLATE 5

E

0 1 2 3 4 5 6 7 8 Feet

DATE: MAY 1946

2



Tracing of portion of original sheet no. 9 of 15 by MICHAEL BAKER, JR., INC. Aug. 1978

SPLIT ROCK POND DAM

GATE CHAMBER SECTION

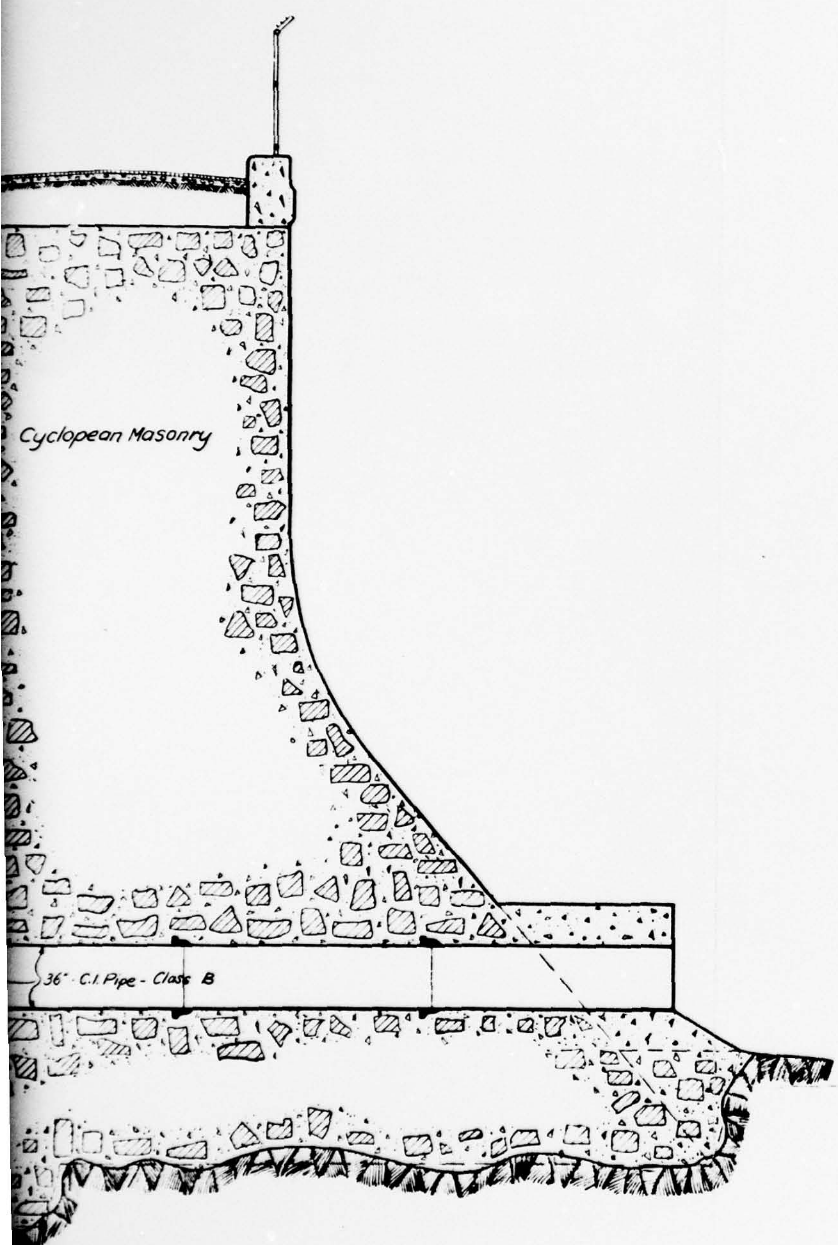


PLATE 6

0 2 4 6 8 Feet

DATE: MAY 1946

2

PHOTOGRAPHS

DETAILED PHOTOGRAPH DESCRIPTIONS

Overall View of Dam - View Upstream (North) at Dam From Left (East) Bank - Gate Chamber Above Dam Crest in Top Right Corner of Photo; 36 Inch Diameter Outlet Pipe in Bottom Right Corner of Photo (Below Gate Chamber); Spillway in Left Side of Photo Behind Trees; Seepage and Mineral Deposits Along Horizontal and Vertical Joints in Downstream Face of Dam - 15 June 1978.

Photo 1 - View West Along Downstream Face of Dam From Left (East) Abutment - Gate Chamber in Center of Photo; Spillway Behind Trees in Left Side of Photo - 15 June 1978.

Photo 2 - View West Along Road Over Crest of Dam From Left (East) Abutment - Gate Chamber in Right Center of Photo - 15 June 1978.

Photo 3 - View Downstream (Southeast) at Left (East) Side of Upstream Face of Dam From Gate Chamber Walkway - Small Trees in Weep Holes of Road Fill Retaining Wall in Top Left Corner of Photo - 16 June 1978.

Photo 4 - View Downstream (Southwest) at Right (West) Side of Upstream Face of Dam From Gate Chamber Walkway - Spillway in Center of Photo - 16 June 1978.

Photo 5 - View Upstream (North) at Spillway From Right (West) Bank - Gneiss Foundation Rock at Bottom of Photo - 15 June 1978.

Photo 6 - View Northeast Across Downstream Face of Dam From Right (West) Abutment - Spillway in Bottom Left Corner of Photo; 36 Inch Diameter Outlet Pipe in Bottom Center of Photo Behind Two Men; Six Inch Diameter Tree Growing From Retaining Wall Weep Hole in Top Center of Photo - 16 June 1978.

Photo 7 - View Upstream (North) at Cracked, Spalled Concrete With Seepage and Mineral Deposits in Downstream Face of Dam - Dam Crest at Top Edge of Photo; Head Wall of 36 Inch Diameter Outlet Pipe in Bottom Center of Photo - 15 June 1978.

Photo 8 - Close-up of Concrete at Approximately El. 830 on Left (East) Side of Downstream Face of Dam - Seepage and Mineral Deposits Along Horizontal Joint; Six Feet Rule for Scale - 15 June 1978.

Photo 9 - View Downstream (South) Over Outlet Channel From Dam Crest Above Outlet Pipe - Concrete Weir (Close-Up in Photo 10) Is at White Water Area in Top Center of Photo - 15 June 1978.

Photo 10 - Close-Up View Upstream at Concrete Weir in Outlet Channel 180 Feet Downstream From Dam (Top Center of Photo 9) - 36 Inch Diameter Outlet Pipe Is Dark Circle in Trees, Left Center of Photo Above Weir - 15 June 1978.

PHOTO 1

PHOTO 2

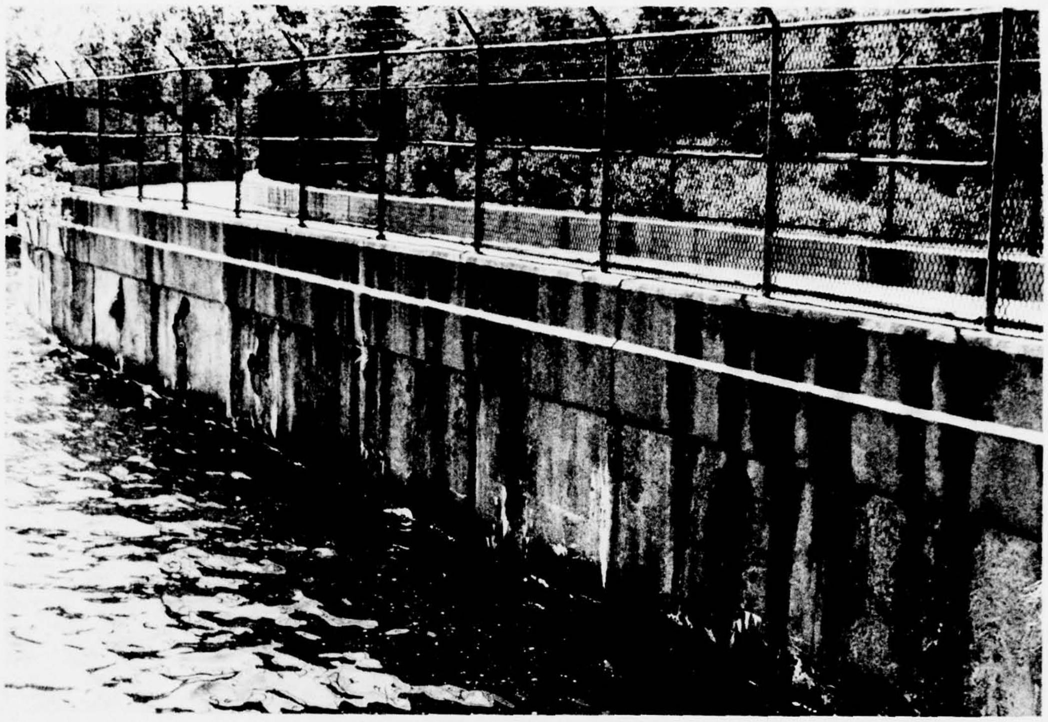


PHOTO 3

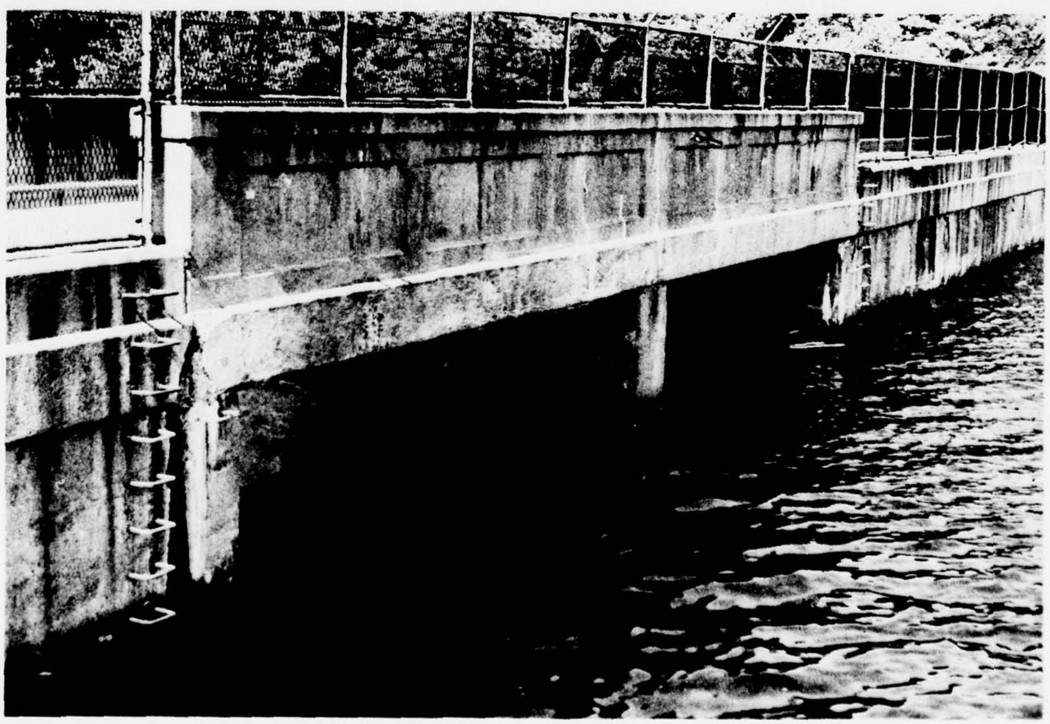


PHOTO 4

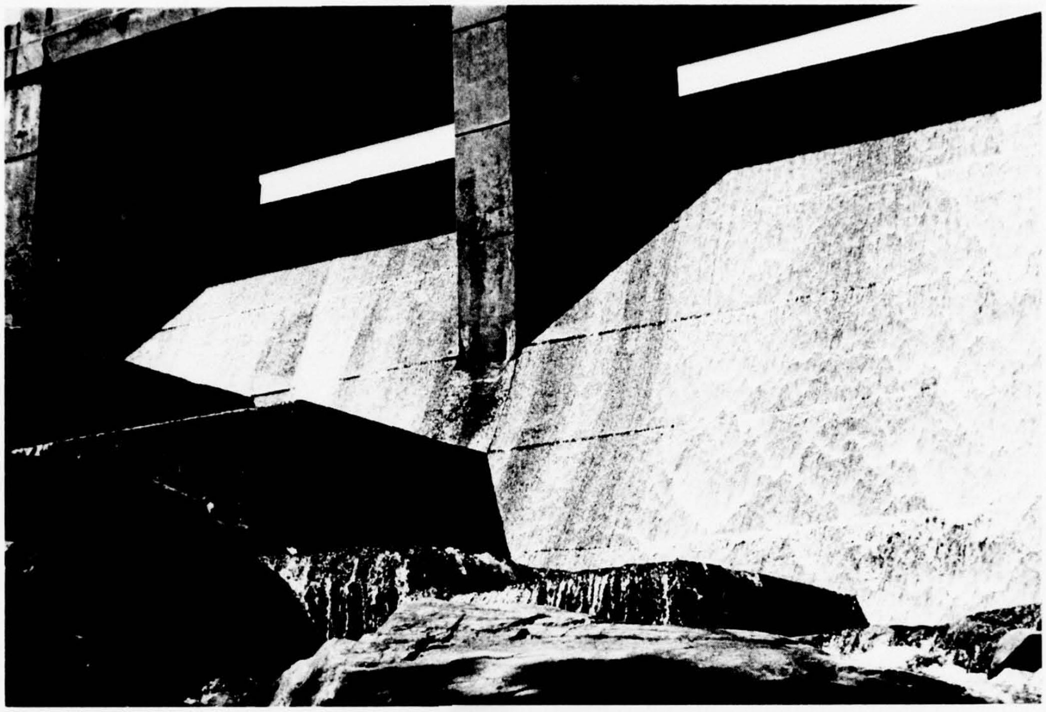


PHOTO 5

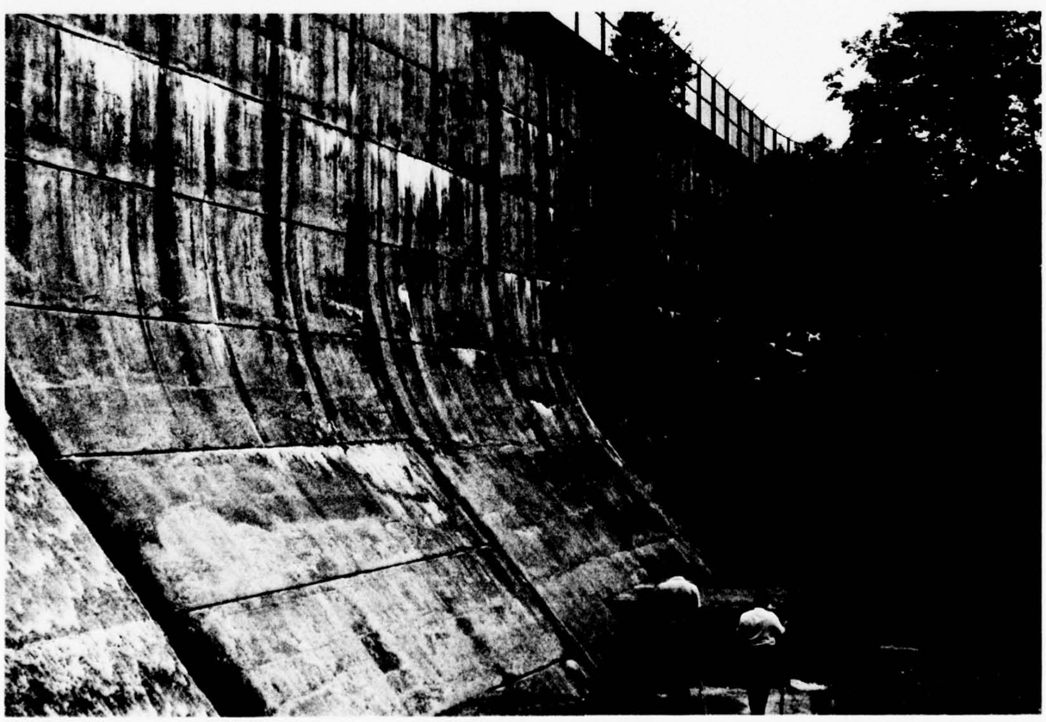


PHOTO 6

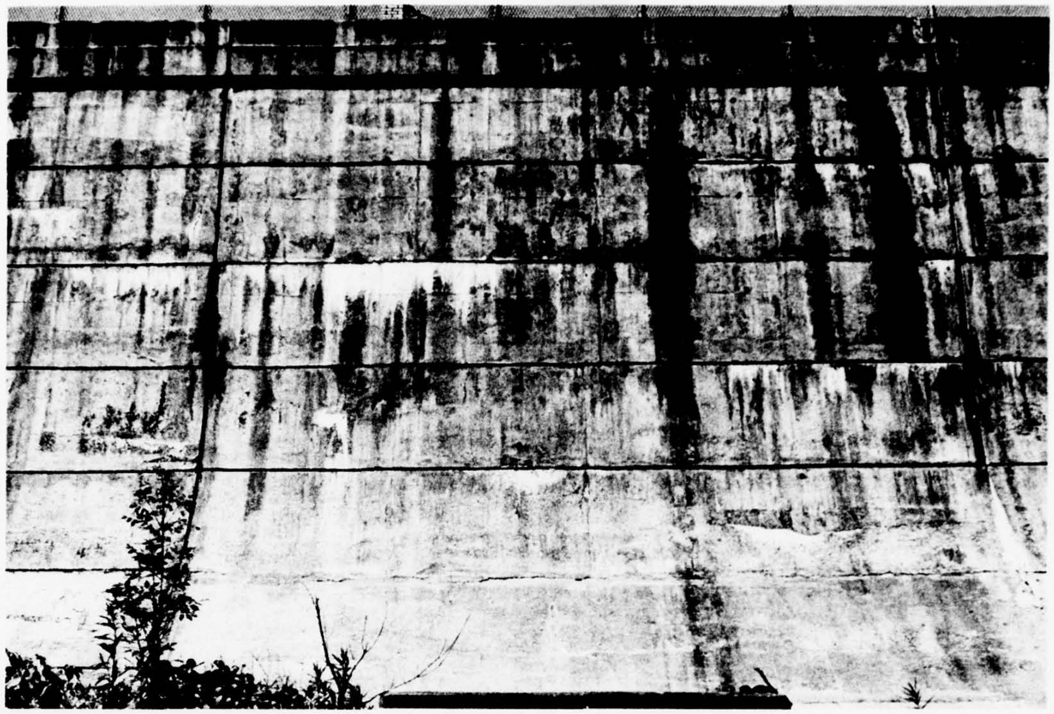


PHOTO 7

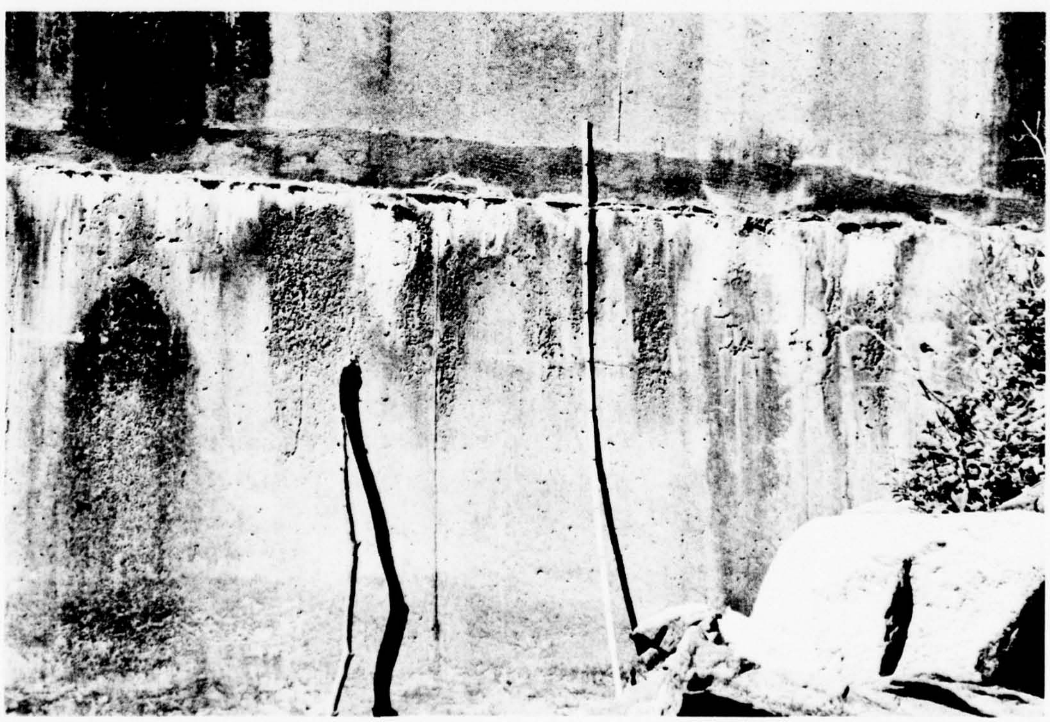


PHOTO 8

PHOTO 9

PHOTO 10

APPENDIX A

CHECK LIST - VISUAL INSPECTION

Check List
Visual Inspection
Phase 1

Name Dam Split Rock Pond Dam County Morris State New Jersey Coordinates Lat. 40° 57.8' Long. 74° 27.6'

Date Inspection 15 and 16 June 1978 Weather Sunny and Mild Temperature 70°F.

Pool Elevation at Time of Inspection 835 M.S.L. Tailwater at Time of Inspection 800± M.S.L.

(No tailwater as such; free discharge from outlet pipe, invert El. 802, to steeply sloping stream channel in gneiss bedrock.)

Inspection Personnel:

MICHAEL BAKER, JR., INC.:	JERSEY CITY PUBLIC WORKS DEPARTMENT: (Part time)
E. U. Gingrich	G. Plastoris
T. J. Dougan	J. Krempa
J. V. Hamel	J. Nelson
	P. Bonno

E. U. Gingrich and J. V. Hamel Recorders

CONCRETE/MASONRY DAMS

Split Rock Pond Dam

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
LEAKAGE	Minor leakage was observed through the horizontal (construction) and vertical (monolith) joints on the left (east) side downstream face of dam--see attached sketch. Minor leakage was observed through jointed rock in both abutments. The top of the leakage was located at approximately El. 820 on the right abutment and approximately El. 800 on the left abutment.	Leakage through concrete requires further investigation. Long term, steady state seepage of approximately one g.p.m. on each abutment is not considered detrimental to structural stability at time of inspection.
STRUCTURE TO ABUTMENT/EMBANKMENT JUNCTIONS	Concrete gravity dam has minor earth fill sections (height less than 10 feet) with concrete gravity retaining walls at both abutments (similar to bridge approach fills). No problems were observed.	
DRAINS	No foundation drains were constructed. Six inch diameter weep holes (some plugged with vegetation and small trees) drain water from subgrade of road along dam crest. Minor to major spalling of concrete was found beneath most of the weep holes on the upstream and downstream sides of dam.	Vegetation should be removed from weep holes to improve drainage and reduce concrete deterioration.
WATER PASSAGES	None were observed.	
FOUNDATION	The foundation is unweathered jointed gneiss exposed in the abutments and in stream channel downstream from the dam.	

CONCRETE/MASONRY DAMS

Split Rock Pond Dam

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SURFACE CRACKS CONCRETE SURFACES	Considerable cracking and spalling was observed on all concrete surfaces, including the downstream face of the dam. Some seepage occurs through cracks and construction joints in the left side of the downstream face of dam--see attached sketch.	The owner should immediately retain a consultant (experienced in stability evaluations for concrete gravity dams) to evaluate the structural stability of the dam as it presently exists and to suggest remedial work as necessary.
STRUCTURAL CRACKING	Considerable cracking was observed in the downstream face of the dam. It was impossible to assess the extent to which the cracks may affect structural integrity from Phase I-type visual inspection. The cracks range in size from one-sixteenth to one-eighth of an inch and are present throughout the vertical face of the dam.	
VERTICAL AND HORIZONTAL ALIGNMENT	There was a slightly bulged area (approximately 0.5 inches outward) noted in left side downstream face of dam approximately El. 830--see attached sketch. The bulged area was approximately three (3) feet high and ten (10) feet wide.	This bulge may be due to pressure of boulders or concrete against form-work during construction of cyclopean masonry, but bulge requires further investigation.
MONOLITH JOINTS	Some cracking, spalling, seepage and mineral deposits were noted along the vertical monolith joints.	
CONSTRUCTION JOINTS	Some cracking, spalling, seepage and mineral deposits were noted along the horizontal construction joints.	

EMBANKMENT

Split Rock Pond Dam

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SURFACE CRACKS	Note: There are minor (height less than 10 feet) earth fill embankments with sections of concrete gravity retaining walls (similar to bridge approach fills) extending from the concrete gravity dam into both abutments. These embankment sections are not exposed to the reservoir water or wave action. No problems were observed in these minor embankment sections.	
UNUSUAL MOVEMENT OR CRACKING AT OR BEYOND THE TOE	No problems were observed.	
SLOUGHING OR EROSION OF EMBANKMENT AND ABUTMENT SLOPES	No problems were observed.	
VERTICAL AND HORIZONTAL ALIGNMENT OF THE CREST	No problems were observed.	
RIPRAP FAILURES	The dam has no riprap. Concrete dam sections and retaining walls provide protection against wave erosion.	

EMBANKMENT

Split Rock Pond Dam

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
JUNCTION OF EMBANKMENT AND ABUTMENT, SPILLWAY AND DAM	No problems were observed.	
ANY NOTICEABLE SEEPAGE	None was observed.	
STAFF GAGE AND RECORDER	Staff gage is at concrete weir located approximately 180 feet downstream from the dam. There is an automatic water level recorder in the gate house; pool levels were recorded daily.	
DRAINS	Six inch diameter weep holes (some plugged with vegetation and small trees) drain the road subgrade through concrete gravity retaining walls. Minor to major spalling of concrete has occurred beneath most of the weep holes.	Vegetation should be removed from the weep holes to improve drainage and reduce concrete deterioration.

OUTLET WORKS

Split Rock Pond Dam

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CRACKING AND SPALLING OF CONCRETE SURFACES IN OUTLET CONDUIT	Outlet conduit is a 36 inch diameter cast-iron pipe located in the base of the left (east) side of the dam.	
INTAKE STRUCTURE	The sluice gates and valves are presently operational. All gates and valves are checked annually and any necessary repairs are made immediately. All sluice gates are operated electronically In the event of a power outage, the gates and valves can be operated manually.	
OUTLET STRUCTURE	Minor spalling of concrete in head wall and downstream slab at pipe outlet was observed.	
OUTLET CHANNEL	No problems were observed.	Gneiss bedrock occurs in steeply sloping stream channel.
EMERGENCY GATE	There is no emergency gate.	

UNGATED SPILLWAY

Split Rock Pond Dam

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONCRETE WEIR	Minor spalling of concrete was found on downstream side of the weir.	
APPROACH CHANNEL	There is no approach channel.	
DISCHARGE CHANNEL	The spillway discharges on concrete slabs. The slab on the left side of the spillway is badly spalled. The slab on the right side of the spillway has minor spalling.	There is hard, unweatherd, gneiss bedrock in a steeply sloping stream channel downstream from the concrete slabs.
BRIDGE AND PIERS	Considerable surficial cracking and spalling were noted on concrete spillway bridge due to winter freeze-thaw action. There was significant concrete cracking on the parapet at both end supports of the bridge.	

GATED SPILLWAY

Split Rock Pond Dam

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONCRETE SILL	Not Applicable	
APPROACH CHANNEL	Not Applicable	
DISCHARGE CHANNEL	Not Applicable	
BRIDGE AND PIERS	Not Applicable	
GATES AND OPERATION EQUIPMENT	Not Applicable	

INSTRUMENTATION

Split Rock Pond Dam

VISUAL EXAMINATION	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
MONUMENTATION/SURVEYS	There are none.	
OBSERVATION WELLS	There are none.	
WEIRS	A curved concrete weir and staff gage are located in the stream about 180 feet downstream from the dam.	The water levels at the weir are obtained by Jersey City Water Department personnel each day and sent to the U.S. Geological Survey on a monthly basis.
PIEZOMETERS	There are none.	
OTHER	A rain gage was reported to exist on nearby private property; readings are taken daily and reported to the U.S. Weather Service. There is an automatic water level gage in the gate house which provides for continuous recording of pool level.	

RESERVOIR

Split Rock Pond Dam

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SLOPES	The slopes are of hard, unweathered, gneiss bedrock with a thin erratic veneer of bouldery glacial till. The slopes are well vegetated and stable from both hydraulic and soil mechanics viewpoints.	
SEDIMENTATION	Sedimentation was inferred to be negligible.	

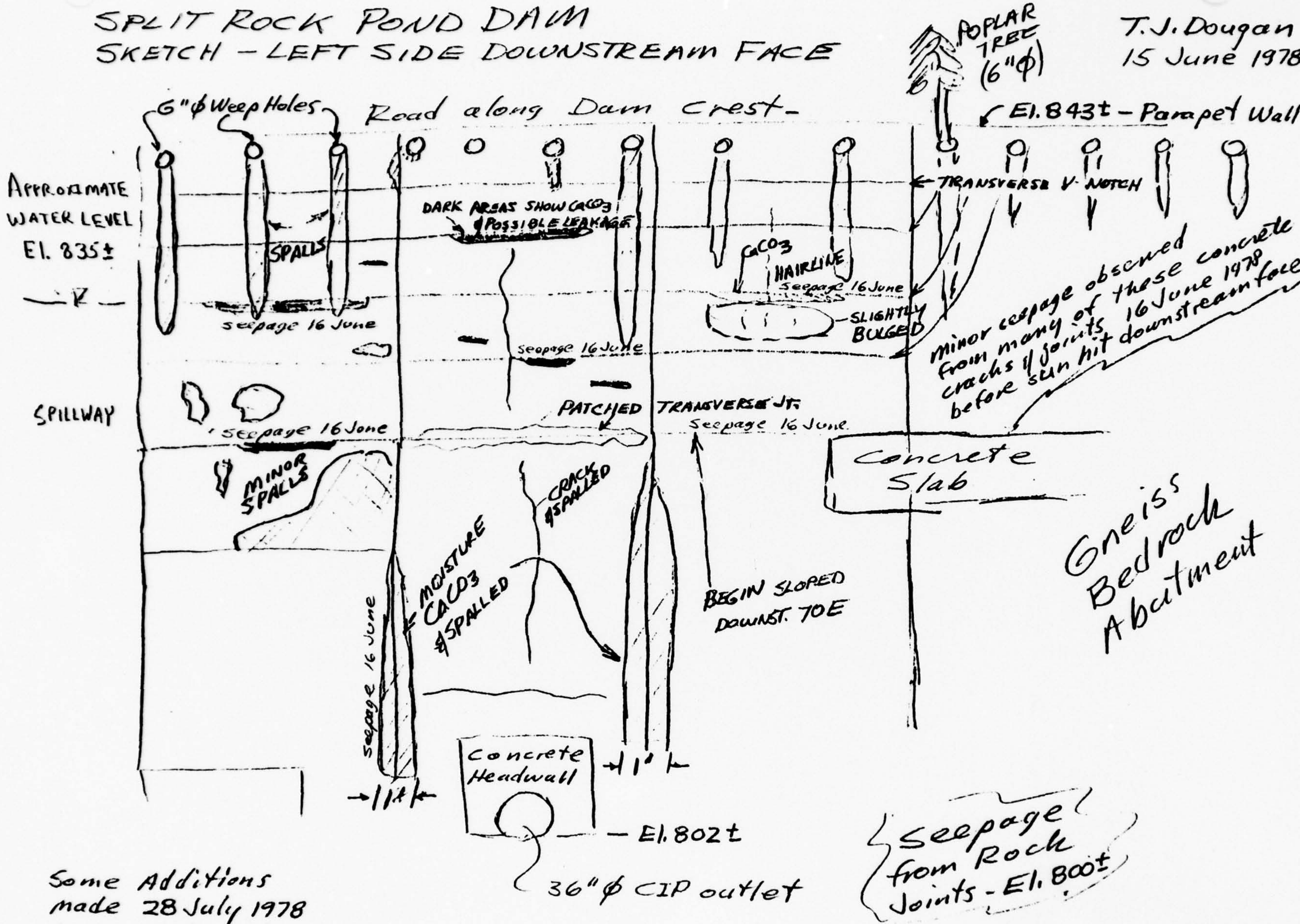
DOWNSTREAM CHANNEL

Split Rock Pond Dam

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONDITION (OBSTRUCTIONS, DEBRIS, ETC.)	Unweathered gneiss bedrock with numerous boulders grading to cobbles and gravel extended about 150 feet downstream from the dam. A gravelly alluvium with boulders extends downstream from the weir located 180 feet downstream from the dam. Some vegetation (weeds, shrubs and trees) and minor debris was observed in the channel.	
SLOPES	Gneiss bedrock with thin erratic vaneer of bouldery glacial till was inferred for the first mile downstream of the dam. Slopes are stable from both geotechnical and hydraulic viewpoints.	
APPROXIMATE NO. OF HOMES AND POPULATION	Reach 1 extends approximately one mile downstream from the dam through an uninhabited narrow valley to the village of Meriden, New Jersey. Reach 2 extends approximately two miles further downstream through a wider, swampy valley from the village of Meriden to the village of Beach Glen. This valley is sparsely populated with an estimated 20 homes and 100 persons in areas which might be affected by flooding in the event of a failure of the dam. (A more detailed survey of Reach 2 would be necessary to establish elevations and population for flooding routing studies.)	

SPLIT ROCK POND DAM
SKETCH – LEFT SIDE DOWNSTREAM FACE

T. J. Dougan
15 June 1978



Some Additions made 28 July 1978 by J. V. Hamel

APPENDIX B

CHECK LIST - ENGINEERING DATA

CHECK LIST
ENGINEERING DATA
DESIGN, CONSTRUCTION, OPERATION

Split Rock Pond Dam

ITEM	REMARKS
PLAN OF DAM	Reference Drawings: "Impounding Reservoir at Split Rock Pond for Jersey City, N.J." prepared by Clyde Potts, Consulting Engineer, New York, New York, September 1923, revised May 1946 (15 sheets--Plans, Sections and Details--prints borrowed from Jersey City Public Works Department; original drawings were lost). Plan of Dam--Reference Drawings--Sheets 1 and 3; Plate 1 of this report.
REGIONAL VICINITY MAP	Reference Drawings--Sheet 1; Section of U.S.G.S. Boonton, New Jersey, 7.5 Minutes Quadrangle in this report as Location Plan.
CONSTRUCTION HISTORY	Limited information in microfiche files of N.J.D.E.P. indicates construction began in late August 1947; work was stopped for the winter in late December 1946; work was resumed in late March 1948; foundation grouting was completed in July 1948; the dam was completed in the autumn of 1948; the dam was dedicated on 19 November 1948.
TYPICAL SECTIONS OF DAM	Reference Drawings--Sheets 4, 5, 6 and 9; Plates 3 and 4 of this report.
HYDROLOGIC/HYDRAULIC DATA	Limited data were available in the microfiche files of N.J.D.E.P.
OUTLETS - PLAN	Reference Drawings--Sheets 1, 3 and 8.
- DETAILS	Reference Drawings--Sheets 8 and 9.
- CONSTRAINTS	None were observed.
- DISCHARGE RATINGS	None were provided by the owner of the dam.
RAINFALL/RESERVOIR RECORDS	Downstream water levels and flow records, pool levels and rainfall records are obtained by the owner. No data were provided for use in this Phase I Inspection Report.

Split Rock Pond Dam

ITEM	REMARKS
DESIGN REPORTS	None were available.
GEOLOGY REPORTS	None were available.
DESIGN COMPUTATIONS HYDROLOGY & HYDRAULICS DAM STABILITY SEEPAGE STUDIES	Very limited design computations are available in the microfiche files of N.J.D.E.P. These are mainly hydrology and hydraulics calculations from review of dam permit application in 1947. The foundation sliding stability analysis in 1947 review of dam permit application indicated a "friction factor" of 0.514 (friction angle of 27^{o}) was required for horizontal sliding stability. No seepage studies were available.
MATERIALS INVESTIGATIONS BORING RECORDS LABORATORY FIELD	None were available.
POST-CONSTRUCTION SURVEYS OF DAM	None were available.
BORROW SOURCES	Concrete gravity dam of cyclopean masonry; sources of concrete aggregate unknown. Boulders for cyclopean masonry were probably obtained from streambed and from local glacial soil deposits. The fill for the dam approach road was probably obtained from local glacial soil deposits.

ITEM	REMARKS
MONITORING SYSTEMS	There are none.
MODIFICATIONS	Minor maintenance work has been completed since construction but no modifications of significance were undertaken since the dam was constructed.
HIGH POOL RECORDS	Records are available in the files of the Jersey City Public Works Department. The highest pool of record reportedly was at El. 836.50; date of this maximum pool level is unknown to Michael Baker, Jr., Inc.
POST-CONSTRUCTION ENGINEERING STUDIES AND REPORTS	None were available.
PRIOR ACCIDENTS OR FAILURE OF DAM DESCRIPTION REPORTS	None have occurred.
MAINTENANCE OPERATION RECORDS	These are available from the Jersey City Public Works Department.

ITEM	REMARKS
SPILLWAY PLAN	Reference Drawings--Sheets 1 and 3; Plate 1 of this report.
SECTIONS	Reference Drawings--Sheets 5 and 6; Plate 4 of this report.
DETAILS	Reference Drawings--Sheets 6 and 7.
OPERATING EQUIPMENT PLANS & DETAILS	Reference Drawings--Sheets 8 and 9.

Note: During discussions with representatives of the Jersey City Public Works Department on 15 June 1978, Michael Baker, Jr., Inc. learned that additional information on the design and construction of Split Rock Pond Dam might exist in the inactive files of the Public Works Department. Baker requested that a search be made to locate such information and, in subsequent telephone conversations, Baker learned that a considerable amount of information does indeed exist. Baker then requested copies of this information in telephone conversations on several occasions, but as of this writing (28 July 1978), no information has yet been received.

CHECK LIST
HYDROLOGIC AND HYDRAULIC DATA
ENGINEERING DATA

DRAINAGE AREA CHARACTERISTICS: 5.6 square miles of wooded, hilly terrain

ELEVATION TOP NORMAL POOL (STORAGE CAPACITY): 835.0 (9517 acre-feet)

ELEVATION TOP FLOOD CONTROL POOL (STORAGE CAPACITY): Not Applicable

ELEVATION MAXIMUM DESIGN POOL: 840.0 (base of spillway bridge)

843.5 (top of crest parapet, maximum structural height)
ELEVATION TOP DAM: 842.0 (low points for overtopping abutment approach roads)

CREST: Spillway weir crest

a. Elevation 835.0
b. Type Concrete ogee
c. Width 50 feet (two 25 feet sections separated by three feet pier)
d. Length Not Applicable
e. Location Spillover Left (west) side of valley
f. Number and Type of Gates Ungated spillway

OUTLET WORKS: Gate chamber with two electrically operated 30 by 30 inch sluice gates

a. Type 36 inch diameter cast-iron pipe
b. Location Right (east) side of base of dam
c. Entrance inverts El. 802.0
d. Exit inverts El. 802.0
e. Emergency draindown facilities Sluice gates and 36 inch pipe

HYDROMETEOROLOGICAL GAGES:

a. Type Rain gage
b. Location Nearby private property
c. Records Reported to U.S. Weather Service

MAXIMUM NON-DAMAGING DISCHARGE 30 m.g.d. equals 31 c.f.s. (Stated by G. Plastoris, Jersey City Public Works Department, during interview on 15 June 1978).

The stream weir and staff gage are located approximately 180 feet downstream from dam; flow rates are obtained daily by the owner and sent to the U.S.G.S. monthly. There is an automatic pool level recorder in the gate house; daily water levels are available from the owner.

Split Rock Pond Dam

APPENDIX C

EXCERPTS FROM
"RECOMMENDED GUIDELINES FOR SAFETY INSPECTION OF DAMS"

SECTION 4.4 - "STABILITY INVESTIGATIONS"

(Pages D-18 through D-27)

Reclamation and Soil Conservation Service. Many other agencies, educational facilities and private consultants can also provide expert advice. Regardless of where such expertise is based, the qualification of those individuals offering to provide it should be carefully examined and evaluated.

4.3.4. Freeboard Allowances. Guidelines on specific minimum freeboard allowances are not considered appropriate because of the many factors involved in such determinations. The investigator will have to assess the critical parameters for each project and develop its minimum requirement. Many projects are reasonably safe without freeboard allowance because they are designed for overtopping, or other factors minimize possible overtopping. Conversely, freeboard allowances of several feet may be necessary to provide a safe condition. Parameters that should be considered include the duration of high water levels in the reservoir during the design flood; the effective wind fetch and reservoir depth available to support wave generation; the probability of high wind speed occurring from a critical direction; the potential wave runup on the dam based on roughness and slope; and the ability of the dam to resist erosion from overtopping waves.

4.4. Stability Investigations. The Phase II stability investigations should be compatible with the guidelines of this paragraph.

4.4.1. Foundation and Material Investigations. The scope of the foundation and materials investigation should be limited to obtaining the information required to analyze the structural stability and to investigate any suspected condition which would adversely affect the safety of the dam. Such investigations may include borings to obtain concrete, embankment, soil foundation, and bedrock samples; testing specimens from these samples to determine the strength and elastic parameters of the materials, including the soft seams, joints, fault gouge and expansive clays or other critical materials in the foundation; determining the character of the bedrock including joints, bedding planes, fractures, faults, voids and caverns, and other geological irregularities; and installing instruments for determining movements, strains, suspected excessive internal seepage pressures, seepage gradients and uplift forces. Special investigations may be necessary where suspect rock types such as limestone, gypsum, salt, basalt, claystone, shales or others are involved in foundations or abutments in order to determine the extent of cavities, piping or other deficiencies in the rock foundation. A concrete core drilling program should be undertaken only when the existence of significant structural cracks is suspected or the general qualitative condition of the concrete is in doubt. The tests of materials will be necessary only where such data are lacking or are outdated.

4.4.2. Stability Assessment. Stability assessments should utilize in situ properties of the structure and its foundation and pertinent geologic information. Geologic information that should be considered includes groundwater and seepage conditions; lithology, stratigraphy, and geologic details disclosed by borings, "as-built" records, and geologic interpretation; maximum past overburden at site as deduced from geologic evidence; bedding, folding and faulting; joints and joint systems; weathering; slickensides, and field evidence relating to slides, faults, movements and earthquake activity. Foundations may present problems where they contain adversely oriented joints, slickensides or fissured material, faults, seams of soft materials, or weak layers. Such defects and excess pore water pressures may contribute to instability. Special tests may be necessary to determine physical properties of particular materials. The results of stability analyses afford a means of evaluating the structure's existing resistance to failure and also the effects of any proposed modifications. Results of stability analyses should be reviewed for compatibility with performance experience when possible.

4.4.2.1. Seismic Stability. The inertial forces for use in the conventional equivalent static force method of analysis should be obtained by multiplying the weight by the seismic coefficient and should be applied as a horizontal force at the center of gravity of the section or element. The seismic coefficients suggested for use with such analyses are listed in Figures 1 through 4. Seismic stability investigations for all high hazard category dams located in Seismic Zone 4 and high hazard dams of the hydraulic fill type in Zone 3 should include suitable dynamic procedures and analyses. Dynamic analyses for other dams and higher seismic coefficients are appropriate if in the judgment of the investigating engineer they are warranted because of proximity to active faults or other reasons. Seismic stability investigations should utilize "state-of-the-art" procedures involving seismological and geological studies to establish earthquake parameters for use in dynamic stability analyses and, where appropriate, the dynamic testing of materials. Stability analyses may be based upon either time-history or response spectra techniques. The results of dynamic analyses should be assessed on the basis of whether or not the dam would have sufficient residual integrity to retain the reservoir during and after the greatest or most adverse earthquake which might occur near the project location.

4.4.2.2. Clay Shale Foundation. Clay shale is a highly overconsolidated sedimentary rock comprised predominantly of clay minerals, with little or no cementation. Foundations of clay shales require special measures in stability investigations. Clay shales, particularly those containing montmorillonite, may be highly susceptible to expansion and consequent loss of strength upon unloading. The shear strength and the resistance to deformation of clay shales may be quite low and high pore water pressures may develop under increase in load. The presence of slickensides in clay shales is usually an indication of low shear strength. Prediction of field behavior of clay shales should not be based solely on results of conventional laboratory tests since they may be misleading. The use of peak shear strengths for clay shales in stability analyses may be unconservative because of nonuniform stress distribution and possible progressive failures. Thus the available shear resistance may be less than if the peak shear strength were mobilized simultaneously along the entire failure surface. In such cases, either greater safety factors or residual shear strength should be used.

4.4.3. Embankment Dams.

4.4.3.1. Liquefaction. The phenomenon of liquefaction of loose, saturated sands and silts may occur when such materials are subjected to shear deformation or earthquake shocks. The possibility of liquefaction must presently be evaluated on the basis of empirical knowledge supplemented by special laboratory tests and engineering judgment. The possibility of liquefaction in sands diminishes as the relative density increases above approximately 70 percent. Hydraulic fill dams in Seismic Zones 3 and 4 should receive particular attention since such dams are susceptible to liquefaction under earthquake shocks.

4.4.3.2. Shear Failure. Shear failure is one in which a portion of an embankment or of an embankment and foundation moves by sliding or rotating relative to the remainder of the mass. It is conventionally represented as occurring along a surface and is so assumed in stability analyses, although shearing may occur in a zone of substantial thickness. The circular arc or the sliding wedge method of analyzing stability, as pertinent, should be used. The circular arc method is generally applicable to essentially homogeneous embankments and to soil foundations consisting of thick deposits of fine-grained soil containing no layers significantly weaker than other strata in the foundation. The wedge method is generally applicable to rockfill dams and to earth dams on foundations containing weak layers. Other methods of analysis such as those employing complex shear surfaces may be appropriate depending on the soil and rock in the dam and foundation. Such methods should be in reputable usage in the engineering profession.

4.4.3.3. Loading Conditions. The loading conditions for which the embankment structures should be investigated are (I) Sudden drawdown from spillway crest elevation or top of gates, (II) Partial pool, (III) Steady state seepage from spillway crest elevation or top of gate elevation, and (IV) Earthquake. Cases I and II apply to upstream slopes only; Case III applies to downstream slopes; and Case IV applies to both upstream and downstream slopes. A summary of suggested strengths and safety factors are shown in Table 4.

TABLE 4

FACTORS OF SAFETY ≠

Case	Loading Condition	Factor of Safety	Shear ≠≠ Strength	Remarks
I	Sudden drawdown from spillway crest or top of gates to minimum drawdown elevation.	1.2*	Minimum composite of R and S shear strengths See Figure 5.	Within the drawdown zone submerged unit weights of materials are used for computing forces resisting sliding and saturated unit weights are used for computing forces contributing to sliding.
II	Partial pool with assumed horizontal steady seepage saturation.	1.5	$\frac{R+S}{2}$ for $R < S$ S for $R > S$	Composite intermediate envelope of R and S shear strengths. See Figure 6.
III	Steady seepage from spillway crest or top of gates with K_h/K_v = 9 assumed**	1.5	Same as Case II	
IV	Earthquake (Cases II and III with seismic loading)	1.0	***	See Figures 1 through 4 for Seismic Coefficients.

≠ Not applicable to embankments on clay shale foundation. Experience has indicated special problems in determination of design shear strengths for clay shale foundations and acceptable safety factors should be compatible with the confidence level in shear strength assumptions.

≠≠ Other strength assumptions may be used if in common usage in the engineering profession.

* The safety factor should not be less than 1.5 when drawdown rate and pore water pressure developed from flow nets are used in stability analyses.

** K_h/K_v is the ratio of horizontal to vertical permeability. A minimum of 9 is suggested for use in compacted embankments and alluvial sediments.

*** Use shear strength for case analyzed without earthquake. It is not necessary to analyze sudden drawdown for earthquake loading. Shear strength tests are classified according to the controlled drainage conditions maintained during the test. R tests are those in which specimen drainage is allowed during consolidation (or swelling) under initial stress conditions, but specimen drainage is not allowed during application of shearing stresses. S tests allow full drainage during initial stress application and shearing is at a slow rate so that complete specimen drainage is permitted during the complete test.

4.4.3.4. Safety Factors. Safety factors for embankment dam stability studies should be based on the ratio of available shear strength to developed shear strength, S_D:

$$S_D = \frac{C}{F.S.} + \sigma \frac{\tan \phi}{F.S.} \tag{1}$$

C = cohesion

ϕ = angle of internal friction

σ = normal stress

The factors of safety listed in Table 4 are recommended as minimum acceptable. Final accepted factors of safety should depend upon the degree of confidence the investigating engineer has in the engineering data available to him. The consequences of a failure with respect to human life and property damage are important considerations in establishing factors of safety for specific investigations.

4.4.3.5. Seepage Failure. A critical uncontrolled underseepage or through seepage condition that develops during a rising pool can quickly reduce a structure which was stable under previous conditions, to a total structural failure. The visually confirmed seepage conditions to be avoided are (1) the exit of the phreatic surface on the downstream slope of the dam and (2) development of hydrostatic heads sufficient to create in the area downstream of the dam sand boils that erode materials by the phenomenon known as "piping" and (3) localized concentrations of seepage along conduits or through pervious zones. The dams most susceptible to seepage problems are those built of or on pervious materials of uniform fine particle size, with no provisions for an internal drainage zone and/or no underseepage controls.

4.4.3.6. Seepage Analyses. Review and modifications to original seepage design analyses should consider conditions observed in the field inspection and piezometer instrumentation. A seepage analysis should consider the permeability ratios resulting from natural deposition and from compaction placement of materials with appropriate variation between horizontal and vertical permeability. An underseepage analysis of the embankment should provide a critical gradient factor of safety for the maximum head condition of not less than 1.5 in the area downstream of the embankment.

$$F.S = i_c/i = \frac{H_c/D_b}{H/D_b} = \frac{D_b\,(\gamma_m - \gamma_w)}{H\,\gamma_w} \qquad (2)$$

i_c = Critical gradient

i = Design gradient

H = Uplift head at downstream toe of dam measured above tailwater

H_c = The critical uplift

D_b = The thickness of the top impervious blanket at the downstream toe of the dam

γ_m = The estimated saturated unit weight of the material in the top impervious blanket

γ_w = The unit weight of water

Where a factor of safety less than 1.5 is obtained the provision of an underseepage control system is indicated. The factor of safety of 1.5 is a recommended minimum and may be adjusted by the responsible engineer based on the competence of the engineering data.

4.4.4. Concrete Dams and Appurtenant Structures.

4.4.4.1. Requirements for Stability. Concrete dams and structures appurtenant to embankment dams should be capable of resisting overturning, sliding and overstressing with adequate factors of safety for normal and maximum loading conditions.

4.4.4.2. Loads. Loadings to be considered in stability analyses include the water load on the upstream face of the dam; the weight of the structure; internal hydrostatic pressures (uplift) within the body of the dam, at the base of the dam and within the foundation; earth and silt loads; ice pressure, seismic and thermal loads, and other loads as applicable. Where tailwater or backwater exists on the downstream side of the structure it should be considered, and assumed uplift pressures should be compatible with drainage provisions and uplift measurements if available. Where applicable, ice pressure should be applied to the contact surface of the structure at normal pool elevation. A unit pressure of not more than 5,000 pounds per square foot should be used. Normally, ice thickness should not be assumed greater than two feet. Earthquake forces should consist of the inertial forces due to the horizontal acceleration of the dam itself and hydrodynamic forces resulting from the reaction of the reservoir water against the structure. Dynamic water pressures for use in conventional methods of analysis may be computed by means of the "Westergaard Formula" using the parabolic approximation (H.M. Westergaard, "Water Pressures on Dams During Earthquakes," Trans., ASCE, Vol 98, 1933, pages 418-433), or similar method.

4.4.4.3. Stresses. The analysis of concrete stresses should be based on in situ properties of the concrete and foundation. Computed maximum compressive stresses for normal operating conditions in the order of 1/3 or less of in situ strengths should be satisfactory. Tensile stresses in unreinforced concrete should be acceptable only in locations where cracks will not adversely affect the overall performance and stability of the structure. Foundation stresses should be such as to provide adequate safety against failure of the foundation material under all loading conditions.

4.4.4.4. Overturning. A gravity structure should be capable of resisting all overturning forces. It can be considered safe against overturning if the resultant of all combinations of horizontal and vertical forces, excluding earthquake forces, acting above any horizontal plane through the structure or at its base is located within the middle third of the section. When earthquake is included the resultant should fall within the limits of the plane or base, and foundation pressures must be acceptable. When these requirements for location of the resultant are not satisfied the investigating engineer should assess the importance to stability of the deviations.

4.4.4.5. Sliding. Sliding of concrete gravity structures and of abutment and foundation rock masses for all types of concrete dams should be evaluated. by the shear-friction resistance concept. The available sliding resistance is compared with the driving force which tends to induce sliding to arrive at a sliding stability safety factor. The investigation should be made along all potential sliding paths. The critical path is that plane or combination of planes which offers the least resistance.

4.4.4.5.1. Sliding Resistance. Sliding resistance is a function of the unit shearing strength at no normal load (cohesion) and the angle of friction on a potential failure surface. It is determined by computing the maximum horizontal driving force which could be resisted along the sliding path under investigation. The following general formula is obtained from the principles of statics and may be derived by resolving forces parallel and perpendicular to the sliding plane:

$$R_R = V \tan(\phi + \alpha) + \frac{cA}{\cos\alpha(1 - \tan\phi \tan\alpha)} \qquad (3)$$

where

R_R = Sliding Resistance (maximum horizontal driving force which can be resisted by the critical path)

ϕ = Angle of internal friction of foundation material or, where applicable, angle of sliding friction

V = Summation of vertical forces (including uplift)

c = Unit shearing strength at zero normal loading along potential failure plane

A = Area of potential failure plane developing unit shear strength "c"

α = Angle between inclined plane and horizontal (positive for uphill sliding)

For sliding downhill the angle α is negative and Equation (1) becomes:

$$R_R = V \tan(\phi - \alpha) + \frac{cA}{\cos\alpha\,(1 + \tan\phi \tan\alpha)} \qquad (4)$$

When the plane of investigation is horizontal, and the angle α is zero and Equation (1) reduced to the following:

$$R_R = V \tan\phi + cA \qquad (5)$$

4.4.4.5.2. Downstream Resistance. When the base of a concrete structure is embedded in rock or the potential failure plane lies below the base, the passive resistance of the downstream layer of rock may sometimes be utilized for sliding resistance. Rock that may be subjected to high velocity water scouring should not be used. The magnitude of the downstream resistance is the lesser of (a) the shearing resistance along the continuation of the potential sliding plane until it daylights or (b) the resistance available from the downstream rock wedge along an inclined plane. The theoretical resistance offered by the passive wedge can be computed by a formula equivalent to formula (3):

$$P_p = W \tan(\phi + \alpha) + \frac{cA}{\cos\alpha\,(1 - \tan\phi \tan\alpha)} \qquad (6)$$

P_p = passive resistance of rock wedge

W = weight (buoyant weight if applicable) of downstream rock wedge above inclined plane of resistance, plus any superimposed loads

ϕ = angle of internal friction or, if applicable, angle of sliding friction

α = angle between inclined failure plane and horizontal

c = unit shearing strength at zero normal load along failure plane

A = area of inclined plane of resistance

When considering cross-bed shear through a relatively shallow, competent rock strut, without adverse jointing or faulting, W and α may be taken at zero and 45°, respectively, and an estimate of passive wedge resistance per unit width obtained by the following equation:

$$P_p = 2\,cD \qquad (7)$$

where

D = Thickness of the rock strut

4.4.4.5.3. Safety Factor. The shear-friction safety factor is obtained by dividing the resistance R_R by H, the summation of horizontal service loads to be applied to the structure:

$$S_{s-f} = \frac{R_R}{H} \quad (8)$$

When the downstream passive wedge contributes to the sliding resistance, the shear fruction safety factor formula becomes:

$$S_{s-f} = \frac{R_R + P_p}{H} \quad (9)$$

The above direct superimposition of passive wedge resistance is valid only if shearing rigidities of the foundation components are similar. Also, the compressive strength and buckling resistance of the downstream rock layer must be sufficient to develop the wedge resistance. For example, a foundation with closely spaced, near horizontal, relatively weak seams might not contain sufficient buckling strength to develop the magnitude of wedge resistance computed from the cross-bed shear strength. In this case wedge resistance should not be assumed without resorting to special treatment (such as installing foundation anchors). Computed sliding safety factors approximating 3 or more for all loading conditions without earthquake, and 1.5 including earthquake, should indicate satisfactory stability, depending upon the reliability of the strength parameters used in the analyses. In some cases when the results of comprehensive foundation studies are available, smaller safety factors may be acceptable. The selection of shear strength parameters should be fully substantiated. The bases for any assumptions; the results of applicable testing, studies and investigations; and all pre-existing, pertinent data should be reported and evaluated.

APPENDIX D

HYDRAULIC/HYDROLOGIC CALCULATIONS

MICHAEL BAKER, JR., INC.
THE BAKER ENGINEERS

Box 280
Beaver, Pa. 15009

Subject SPLIT ROCK DAM S.O. No. ____
Sheet No. ____ of ____
Drawing No. ____
Computed by TWS Checked by ____ Date 8-30-78

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Subject N.J. Dam Inspection S.O. No.
Split Rock Pond Sheet No. 1 of 49
Rainfall - Runoff for PMF Drawing No.
Computed by REK Checked by TWS Date 7/27/78

Using Design of Small Dams and Rainfall Distribution from EM-1110-2-163

Time	Cum Rainfall	Inc Rainfall	Cumul. Excess (Q)	Δ Excess	Inc Loss	Min Loss	Adj Excess
0.0	0.0						
0.5	1.0	1.0	0.00 ✓	0.00	1.00		0.00
1.0	2.0	1.0	0.09 ✓	0.09	0.91		0.09
1.5	3.2	1.2	0.48 ✓	0.39	0.81		0.39
2.0	4.4	1.2	1.08 ✓	0.60	0.60		0.60
2.5	5.9	1.5	2.02 ✓	0.94	0.56		0.94
3.0	7.4	1.5	3.10 ✓	1.08	0.42		1.08
3.5	11.2	3.8	6.17 ✓	3.07	0.73		3.07
4.0	15.0	3.8	9.53 ✓	3.36	0.44		3.36
4.5	16.4	1.4	10.80 ✓	1.27	0.13		1.27
5.0	17.8	1.4	12.10 ✓	1.30	0.10	0.12	1.28
5.5	18.9	1.1	13.12 ✓	1.02	0.08	0.12	0.98
6.0	20.0	1.1	14.15 ✓	1.03	0.07	0.12	0.98
							14.04

$$Q = \frac{(P-0.2S)^2}{P+0.8S} = \frac{(P-1.23)^2}{P+4.90}$$

$$S = \frac{1000}{CN} - 10 = \frac{1000}{62} - 10 = 6.13$$

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Box 280
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Subject SPLIT ROCK POND
100 RAINFALL DURATION

S.O. No. ______
Sheet No. 2 of 49
Drawing No. ______

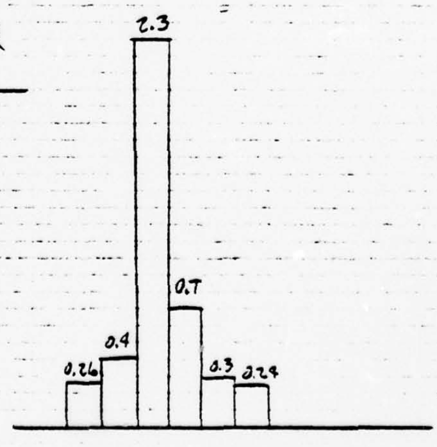
Computed by TWS Checked by RE? Date 8-29-78

Rainfall obtained from Technical Paper 40 for various durations

Duration	Inches
5 min	0.85
10 min	1.31
15 min	1.66
30 min	2.3
1 hr	3.0
2 hr	3.7
3 hr	4.2
6 hr	5.2
12 hr	6.2
24 hr	7.2

T_c = 2.08 hours

Time	Inc. Rainfall	Rearranged Rainfall
0.5	2.3	0.26
1.0	0.7	0.40
1.5	0.4	2.30
2.0	0.3	0.70
2.5	0.26	0.30
3.0	0.24	0.24



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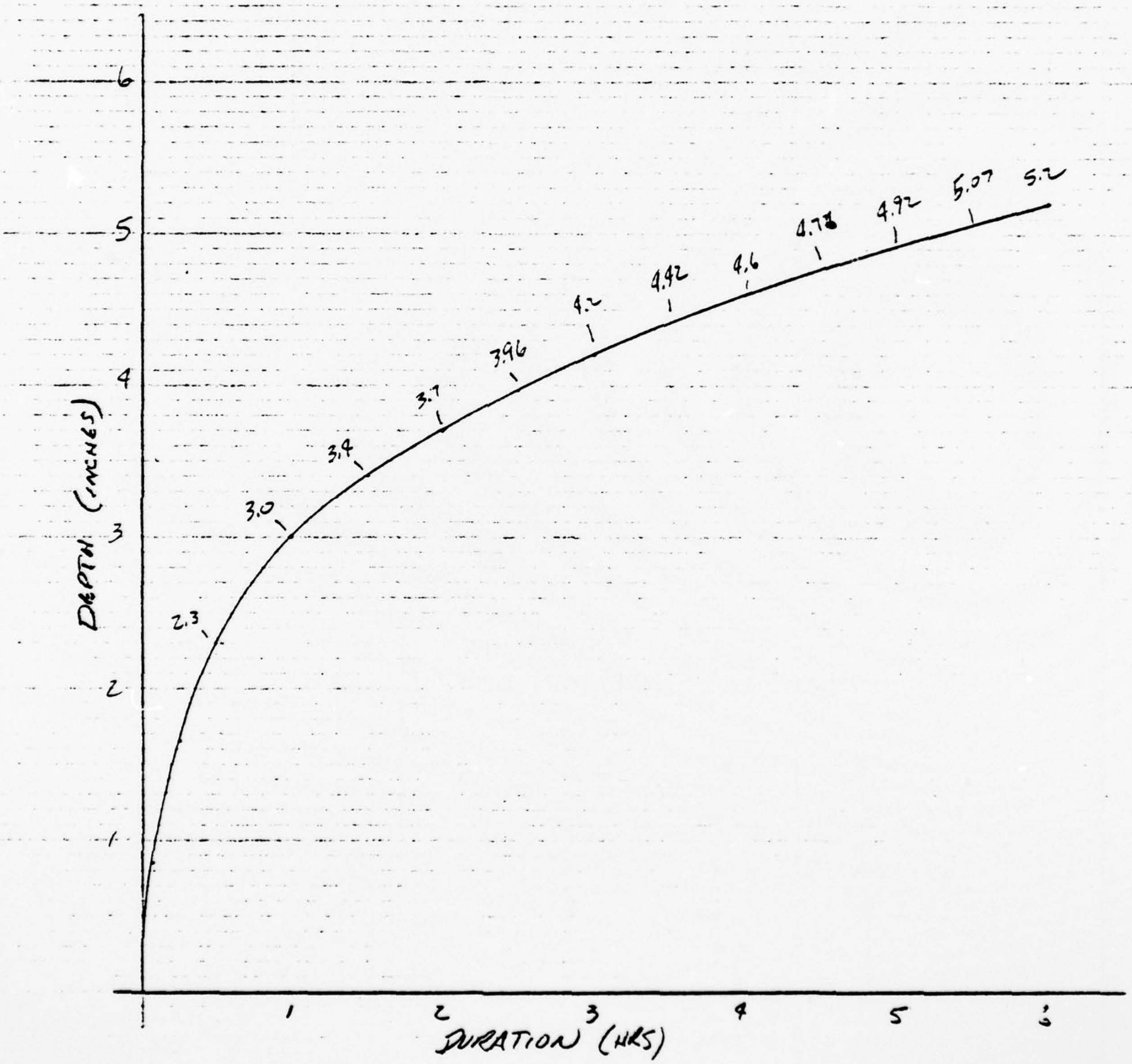
Subject: SPLIT ROCK POND S.O. No.:
Sheet No. 3 of 49
RAINFALL DURATION CURVE Drawing No.:
Computed by: TWS Checked by: ZEH Date: 8-29-78




N
A
SPLIT ROCK RESERVOIR
ROCKAWAY TWP, PASSAIC Co
NEW JERSEY
1" = 2000'
D.A.
5.59 sq mi

sheet 4 of 49

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Box 280
Beaver, Pa. 15009

Subject SPLIT ROCK POND RES. S.O. No.
CN VALUES Sheet No. 5 of 49
Drawing No.
Computed by J. SAWYER Checked by REB Date 7-23-78

TOTAL AREA = 38.96 in^2 = 5.59 mi^2 RESEVOIR = 5.70

LAND USE	AREA IN^2	SOIL CLASS	%	CURVE #	PRODUCT
WOODS	29.85	B	89.7	60	5382.0
MEADOWS	1.21	B	3.6	58	208.8
ROADS					
DIRT	.16	B	0.5	82	41.0
HARD	.07	B	0.2	84	16.8
WATER SURFACE	1.65	B	5.0	100	500.0
RESIDENTIAL 1/2 ACRE	.32	B	1.0	70	70.0
	33.26				6218.6

CN = 62

"DESIGN OF SMALL DAMS" p. 70

ROUTE A

1 TO 2	OVERLAND PASTURES	250' @ 4% SLOPE	V=3.0	1.4 M
2 TO 3	LAKE	750' @ " "	V=3.0	4.2
3 TO 4	WOODED SWAMP (NATURAL CHANNEL NOT WELL DEFINED) 945-920	3750' @ 1% SLOPE	V=1.0	62.5
4 TO 5	WOODED NATURAL CHANNEL 920-880	1900' @ 2% SLOPE	V=2.0	15.8
5 TO 6	NATURAL CHANNEL 880-835	3700' @ 1% SLOPE	V=1.5	41.1
				125.00'
				T_c = 2.08 HR

ROUTE B

BY INSPECTION OF QUAD SHORTER THAN ROUTE A

ROUTE C

6000' 1072-835 4% SLOPE V=3.5 T_c=28 MIN (LOW)

ROUTE D

5500' 1050-835 4% SLOPE V=3.5 T_c=26 MIN (LOW)

MICHAEL BAKER, JR., INC.
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Box 280
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Subject NJ DAM INSPECTION S.O. No.
SPLIT ROCK POND DAM Sheet No. 7 of 17
UNIT GRAPH PARAMETERS Drawing No.
Computed by RCH Checked by REK Date 7/27/78

FOR $T_c = 2.08$ HR USING $D = 30$ MIN $= .5$ HR.

$$T_P = D/2 + .6\, T_c = .5/2 + .6(2.08) = \underline{\underline{1.5 \text{ HRS.}}}$$

Computed from Design of Small Dams and SCS Hydrology Handbook Ch 4

$$q_P = \frac{484\, A Q}{T_P} = \frac{484\ (4.73)(1.0)}{1.5} = 1526 \text{ CFS}$$

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Box 280
Beaver, Pa. 15009

Subject N.J. Dam Inspection S.O. No.
Unit Hydrograph - Split Sheet No. 8 of 49
Fork Pond for Land Area (9.73 sq. mi.) Drawing No.
Computed by REH Checked by TWS Date 7/27/78

Design of Small Dams - SCS Curvilinear Hydrograph

Time	T/Tp	q/qp	q	qrev.
0	0.00	0.00	0	0
0.5	0.33	0.18	275	271
1.0	0.67	0.72	1099	1084
1.5	1.00	1.00	1526	1505
2.0	1.33	0.82	1251	1233
2.5	1.67	0.51	778	767
3.0	2.00	0.32	488	481
3.5	2.33	0.197	301	297
4.0	2.67	0.117	178	175
4.5	3.00	0.075	114	112
5.0	3.33	0.047	72	71
5.5	3.67	0.030	46	45
6.0	4.00	0.019	29	29
6.5	4.33	0.012	18	18
7.0	4.67	0.006	9	9
7.5	5.00	0.004	6	6
8.0	5.33	0.001	2	2
8.5	5.67	0.00	0	0

6192 cfs

= 1.014" rainfall

MICHAEL BAKER, JR., INC.
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Box 280
Beaver, Pa. 15009

Subject SPLIT ROCK POND S.O. No.

UNIT GRAPH FOR LAKE AREA (0.86 S.M.)

Drawing No.

Computed by TWS Checked by RE Date 8-29-78

UNITGRAPH FOR LAKE AREA = 0.86 square miles

For a duration = 0.5 hours

determine the amount of flow fir 1 inch of runoff

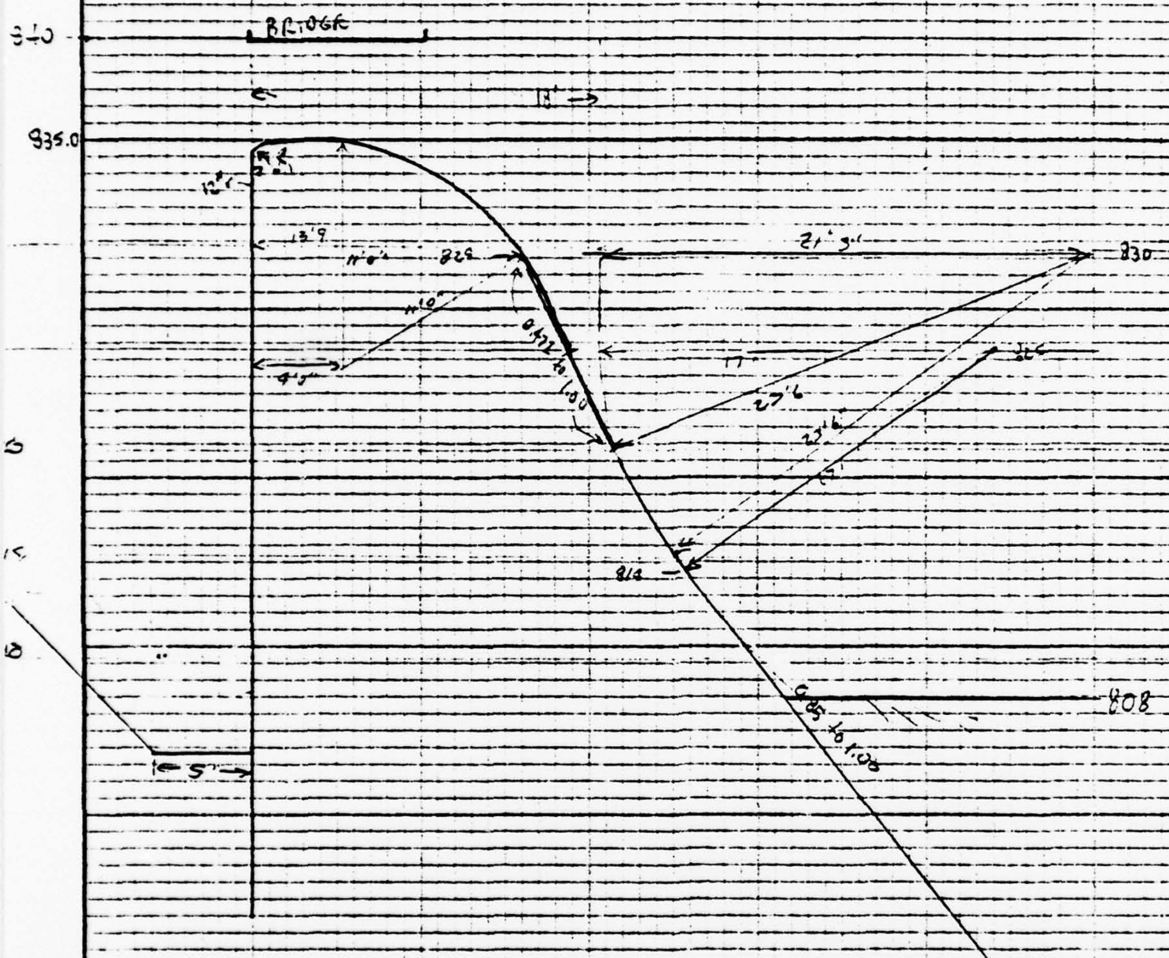
$$\frac{1 \text{ inch}}{30 \text{ minute}} \times 0.86 \text{ miles}^2 \times \frac{5280^2 ft^2}{mi^2} \times \frac{1 \text{ min}}{60 \text{ sec}} \times \frac{1 \text{ ft}}{12 \text{ in}} = 1110 \text{ cfs}$$

∴ Lake Unitgraph = 1110 cfs/in

MICHAEL BAKER, JR., INC.
THE BAKER ENGINEERS

Box 280
Beaver, Pa. 15009

Subject SPLIT ROCK POND
SPILLWAY SECTION - OGEE
S.O. No.
Sheet No. 10 of 49
Drawing No.
Computed by TWS
Checked by
Date 7-20-78



$R_1 = 0.253 H_0$

$12 = 0.253 H_0$

$H_0 = 47.4''$

$= 3.95'$

MICHAEL BAKER, JR., INC.
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Box 280
Beaver, Pa. 15009

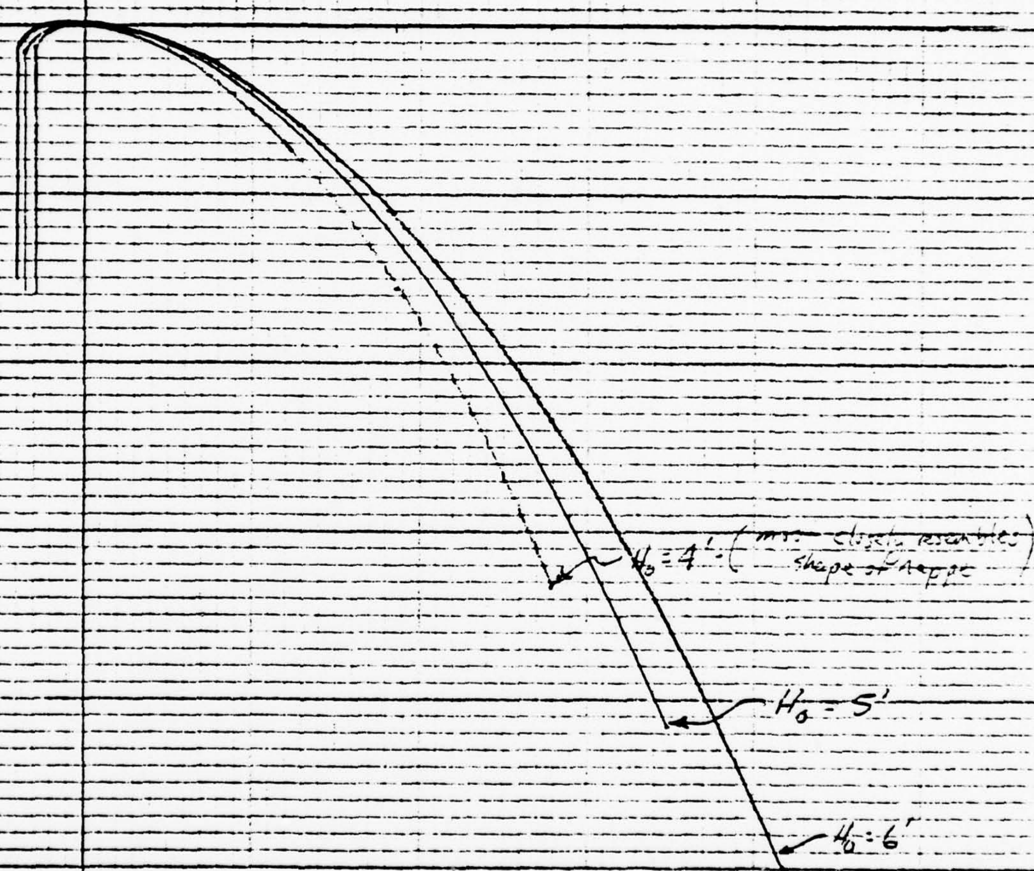
Subject SPLIT ROCK POND S.O. No.

SAMPLE OGEE CREST w/ H_o Drawing No.

Computed by TWS Checked by Date 7-20-78

Scale 1/4 = 1' fig 249 "Design of Small Dams"

approach velocity considered negligible because $P \gg H$ 30 > 5



MICHAEL BAKER, JR., INC.
THE BAKER ENGINEERS

Box 280
Beaver, Pa. 15009

Subject: SPLIT ROCK POND
OGEE RATING

S.O. No.
Sheet No. 12 of 49
Drawing No.
Computed by TWS Checked by PEK Date 7-20-78

Design of Small Dams

$Q = CLH^{3/2}$ H_e of 0 = 835.0 msl

H_e	H_o	$\frac{H_e}{H_o}$	C/C_o	C_o*	C	L	$H^{3/2}$	Q (cfs)	Elev
0.5	4.0	0.125	0.83	3.95	3.28	50	0.354	58	835.0
1.0		0.25	0.865		3.42		1.0	171	836.0
1.5		0.375	0.895		3.54		1.84	326	
2.0		0.50	0.92		3.63		2.83	514	837.0
2.5		0.625	0.945		3.73		3.95	737	
3.0		0.750	0.965		3.81		5.20	991	838.0
3.5		0.875	0.985		3.89		6.55	1274	
4.0		1.00	1.0		3.95		8.0	1580	839.0
4.5		1.125	1.015		4.01		9.54	1913	
5.0		1.25	1.03		4.07		11.20	2279	840.0
6.0								2440	841.0
7.0								2724	842.0

orifice flow

w/ road Bridge

from page 14

* $P/H_o > 3.0 \therefore C_o = 3.95$

MICHAEL BAKER, JR., INC.
THE BAKER ENGINEERS

Box 280
Beaver, Pa. 15009

Subject: Solid Rock Pond
Spillway Rating

S.O. No.

Drawing No.

Computed by REH Checked by TWS Date 7/31/73

Using a design head of 3.0' results in an increased spillway capacity

Original spillway rating was developed using the equation $Q = 3.3LH^{3/2}$. The discharge coefficient of 3.3 is relatively low which is reflected in our present spillway rating in which coefficients ranging from 3.38 to 4.07 were used

MICHAEL BAKER, JR., INC.
THE BAKER ENGINEERS

Box 280
Beaver, Pa. 15009

Subject SPLIT ROCK POND — S.O. No.
ORIFICE COMPS — Sheet No. 14 of 49
Drawing No.
Computed by RCH — Checked by REK — Date 8/24/78

ORIFICE FLOW: FROM - "HANDBOOK OF HYDRAULICS"
HORACE W. KING ET AL, 1963
McGRAW-HILL, PG 4-3 / 4-5

$Q = Ca\sqrt{2gh_o}$ EQ 4-10

FOR
$H_o = 3.5$ $Q = .65(250)\sqrt{2(32.2)(3.5)} = 2440$ CFS ✓
841 MSL

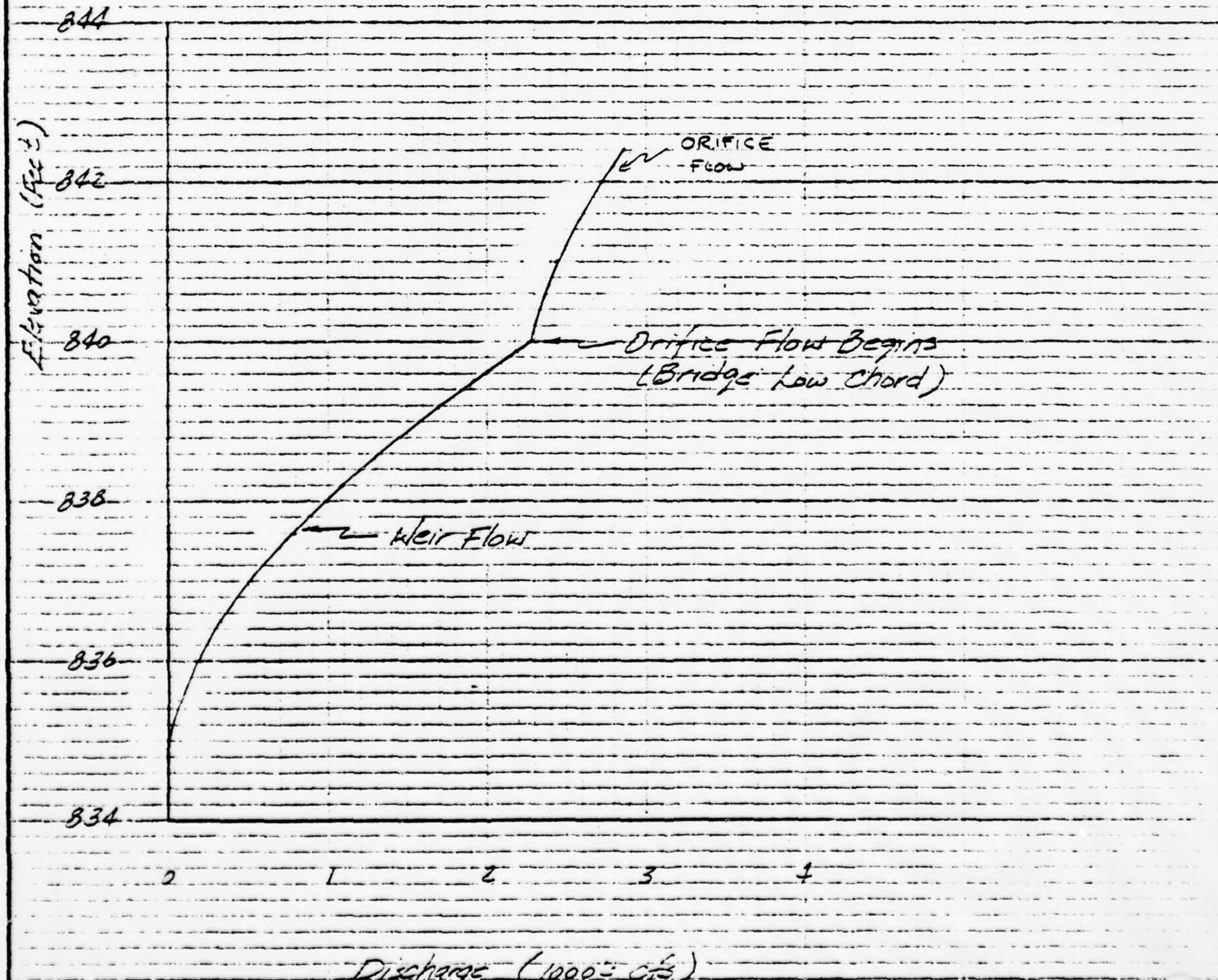
FOR
$H_o = 4.5$ $Q = .64(250)\sqrt{2(32.2)(4.5)} = 2724$ CFS ✓
842 MSL

*COEFFICIENTS OF DISCHARGE USED WERE OBTAINED FROM TABLE 4-5 PG. 4-31 OF KINGS "HANDBOOK OF HYDRAULICS" BASED ON EXPERIMENTS CONDUCTED BY H.T. BOVEY, PUBLISHED IN "HYDRAULICS", H.T. BOVEY, p. 40, JOHN WILEY & SONS, INC., NEW YORK, 1909. THESE FIGURES WERE OBTAINED FROM AN EXPERIMENTAL ORIFICE MUCH SMALLER IN SIZE BUT HAVING THE SAME CROSS SECTION SHAPE AS THE SUBJECT ORIFICE.

MICHAEL BAKER, JR., INC.
THE BAKER ENGINEERS

Box 280
Beaver, Pa. 15009

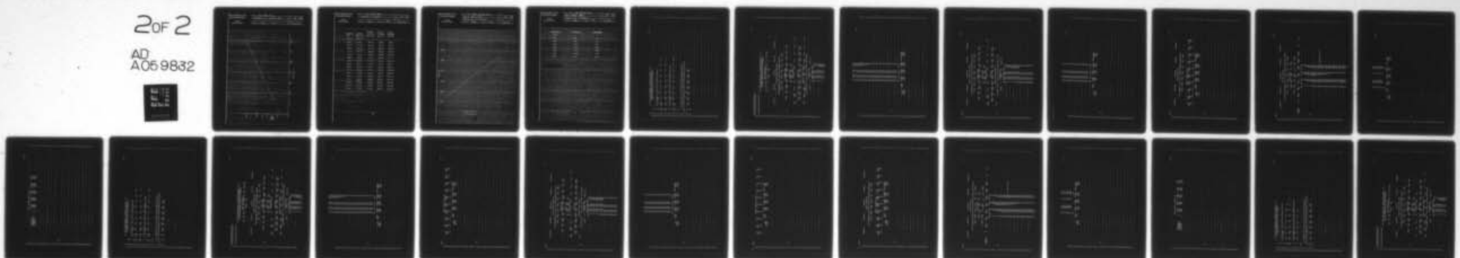
Subject N.J. Dam Inspection
Split Rock Pond
Spillway Rating Curve
S.O. No.
Sheet No. 15 of 49
Drawing No.
Computed by REH
Checked by TWS
Date 7/27/78



AD-A059 832 BAKER (MICHAEL) JR INC BEAVER PA F/G 13/2
NATIONAL DAM SAFETY PROGRAM. SPLIT ROCK POND DAM (NJ00264), PAS--ETC(U)
AUG 78 M BAKER DACW61-78-C-0141

UNCLASSIFIED NL

2 OF 2
AD
A059832



END
DATE
FILMED
12-78
DDC

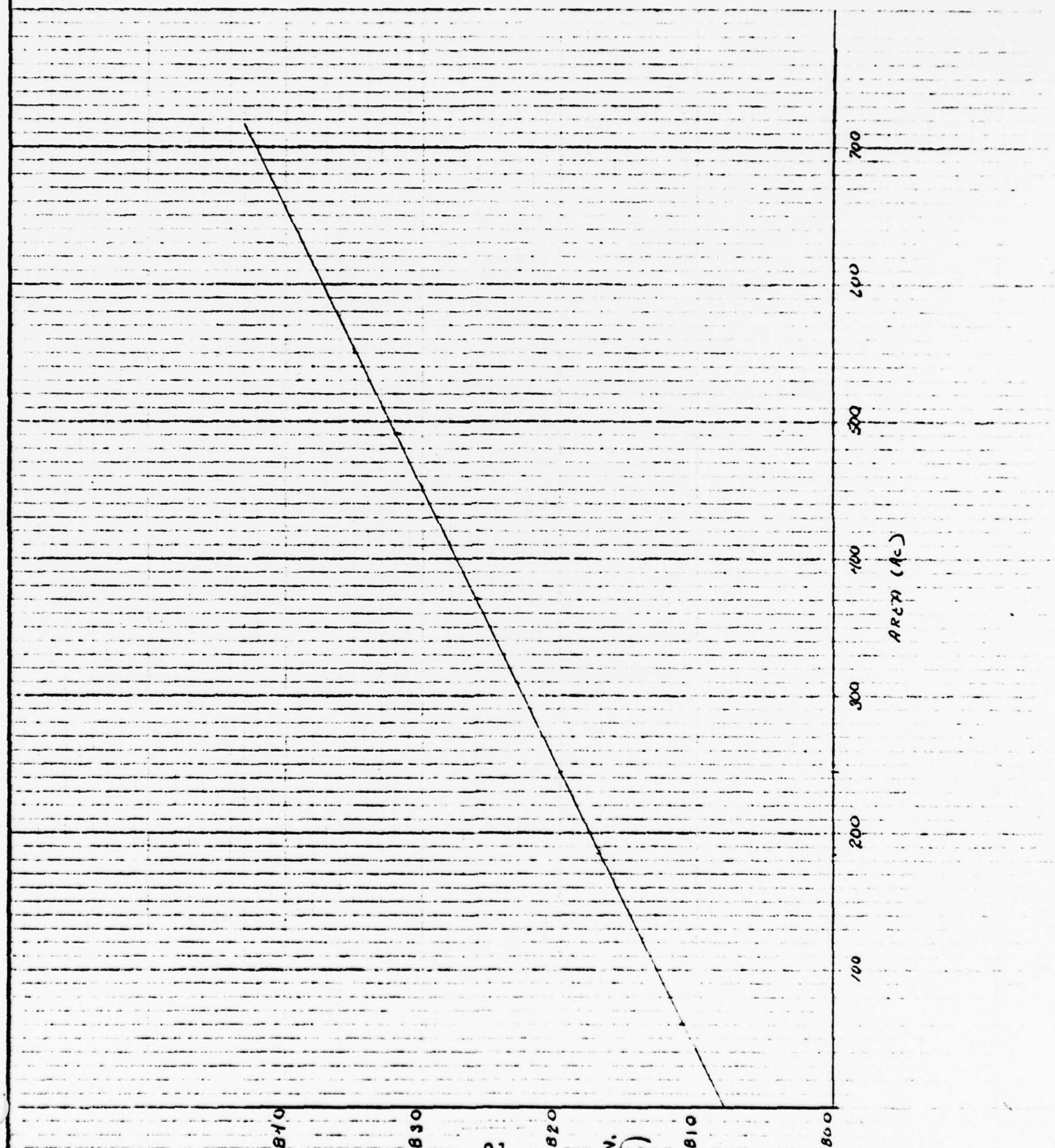
MICHAEL BAKER, JR., INC.
THE BAKER ENGINEERS

Box 280
Beaver, Pa. 15009

Subject SPLIT ROCK POND
ELEVATION VS SURFACE AREA

S.O. No.
Sheet No. 16 of 49
Drawing No.

Computed by ALR Checked by Date 02/25/78



MICHAEL BAKER, JR., INC.
THE BAKER ENGINEERS

Box 280
Beaver, Pa. 15009

Subject SPLIT ROCK POND
ELEVATION VS STORAGE
S.O. No.
Sheet No. 17 of 49
Drawing No.
Computed by ALB Checked by TWS Date 07/25/78

ELEVATION (FT)	SURFACE AREA (AC)	AVERAGE SURFACE AREA (AC)	INCREM. STORAGE (AC-FT)	CUMUL STORAGE (AC-FT)
835.0	550.98			
835.5	561.18	556.08	278.04	278.04
836.0	571.39	566.29	283.14	561.18
836.5	581.59	576.49	288.25	849.43
837.0	591.79	586.69	293.35	1142.77
837.5	602.00	596.90	298.45	1441.22
838.0	612.20	607.10	303.55	1744.77
838.5	622.40	617.30	308.65	2053.42
839.0	632.61	627.51	313.75	2367.17
839.5	642.81	637.71	318.86	2686.03
840.0	653.01	647.91	323.96	3009.98
840.5	663.22	658.12	329.06	3339.04
841.0	673.42	668.32	334.16	3673.20
841.5	683.62	678.52	339.26	4012.46
842.0	693.83	688.73	344.36	4356.82
842.5	704.03	698.93	349.47	4706.29
843.0	714.23	709.13	354.57	5060.85

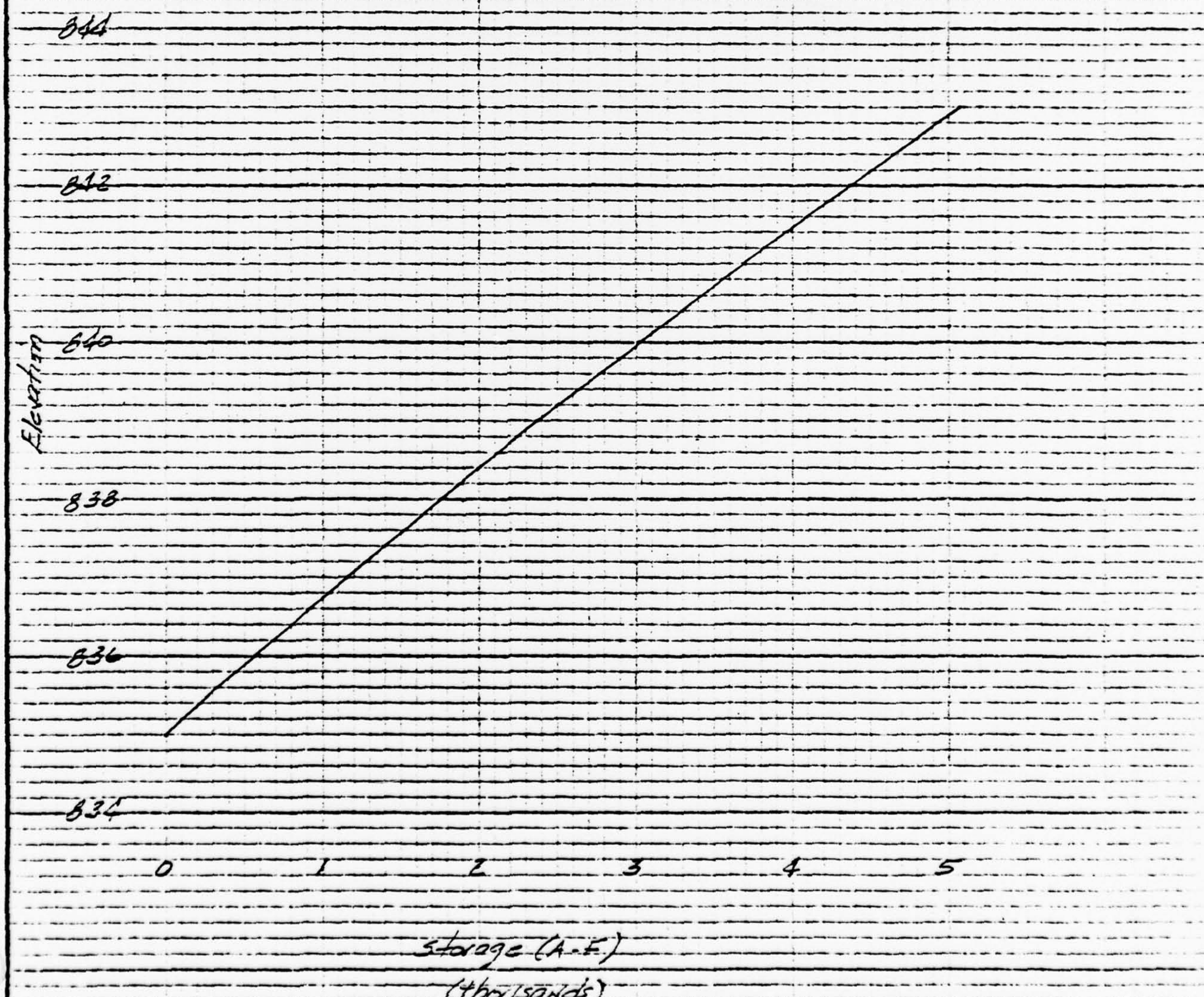
MICHAEL BAKER, JR., INC.
THE BAKER ENGINEERS

Box 280
Beaver, Pa. 15009

Subject N.J. Dam Inspection
Split Rock Pond
Elevation vs. storage

S.O. No.
Sheet No. 18 of 49
Drawing No.

Computed by REH Checked by TWS Date 7/27/78



MICHAEL BAKER, JR., INC.
THE BAKER ENGINEERS

Box 280
Beaver, Pa. 15009

Subject N.J. Dam Inspection S.O. No.
Split Rock Pond Sheet No. 19 of 49
Storage vs. Discharge Drawing No.
Computed by REH Checked by TWS Date 7/27/78

Elevation	Storage	Discharge
835	2	0
836	561	171
837	1143	514
838	1745	991
839	2367	1580
840	3010	2279
841	3673	2440
842	4350	2724

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A             NEW JERSEY DAM INSPECTIONS - PHILADELPHIA DISTRICT C.O.E.
A                        SPLIT ROCK POND DAM (SPLITROCK RESERVOIR)
A                          PROBABLE MAXIMUM STORM BY UNIT HYDROGRAPHS
B      48                  30
L       3
K       0         1                             2         1         1
1                THIS HYDROGRAPH IS FOR SPLIT ROCK WATERSHED LAND AREA
M       0        -1       4.73       0
O      12
1       0      0.09      0.39      0.60      0.94      1.08      3.07      3.36      1.27      1.28
1    0.98      0.98
T
U      17
1     271      1084      1505      1233       767       481       297       175       112        71
1      45        29        18         9         6         2         0
Y                           1
K       0         2                             2         1         1
U                THIS HYDROGRAPH IS FOR SPLIT ROCK POND WATERSHED LAKE AREA
M       0        -1       .86         0
O      12
1       0      0.09      0.39      0.60      0.94      1.08      3.07      3.36      1.27      1.28
1    0.98      0.98
T                                                                                               1
U       1
1    1110
Y                           1
K       2         3                             2         1         1
1              THIS HYDROGRAPH IS FOR SPLIT ROCK POND WATERSHED COMBINED AREA
K       1         4                             2         1         1
1   96      THIS IS OUTFLOW HYDROGRAPH OF SPLIT ROCK POND WATERSHED AFTER ROUTING
Y                           1         1
1       1
2       0       561      1143      1745      2367      3010      3673      4350
3       0       171       514       991      1580      2279      2440      2724
K      99
A
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******************************
HEC-1 VERSION DATED JAN 1973

******************************
                        NEW JERSEY DAM INSPECTIONS - PHILADELPHIA DISTRICT C.O.E.
                             SPLIT ROCK POND DAM (SPLITROCK RESERVOIR)
                              PROBABLE MAXIMUM STORM BY UNIT HYDROGRAPHS

                                     JOB SPECIFICATION
                         NQ    NHR  NMIN  IDAY   IHR  IMIN METRC  IPLT  IPRT NSTAN
                         48      0    30     0     0     0     0     0     0     0
                                          JOPER    NWT
                                              3      0

      **********         **********         **********         **********         **********

                                  SUB-AREA RUNOFF COMPUTATION

                        THIS HYDROGRAPH IS FOR SPLIT ROCK WATERSHED LAND AREA

                           ISTAQ   ICOMP   IECON   ITAPE    JPLT    JPRT   INAME
                               1       0       0       0       2       1       1

                                       HYDROGRAPH DATA
                IHYDG   IUHG  TAREA   SNAP  TRSDA  TRSPC  RATIO  ISNOW  ISAME  LOCAL
                    0     -1   4.73    0.0    0.0    0.0    0.0      0      0      0

                                         PRECIP DATA
                                   NP   STORM    DAJ     DAK
                                   12     0.0    0.0     0.0
                                        PRECIP PATTERN
      0.0      0.09      0.39      0.60      0.94      1.08      3.07      3.36      1.27      1.28
      0.98     0.98

                                          LOSS DATA
               STRKR  DLTKR  RTIOL  ERAIN  STRKS  RTIOK  STRTL  CNSTL  ALSMX  RTIMP
                 0.0    0.0   1.00    0.0    0.0   1.00    0.0    0.0    0.0    0.0

                                   GIVEN UNIT GRAPH, NUHGQ=  17
      271.     1084.     1505.     1233.      767.      481.      297.      175.      112.       71.
       45.       29.       18.        9.        6.        2.        0.
                        UNIT GRAPH TOTALS    6105. CFS OR 1.00 INCHES OVER THE AREA

                                        RECESSION DATA
                            STRTQ=    0.0     QRCSN=    0.0     RTIOR= 1.00

                                       END-OF-PERIOD FLOW
                                   TIME   RAIN   EXCS    COMP Q
                                      1    0.0    0.0        0.
                                      2   0.09   0.09       24.
                                      3   0.39   0.39      203.
                                      4   0.60   0.60      721.
                                      5   0.94   0.94     1603.
                                      6   1.08   1.08     2765.
                                      7   3.07   3.07     4500.
                                      8   3.36   3.36     7697.
                                      9   1.27   1.27    11080.
```

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11	0.98	0.98	11016.
12	0.98	0.98	9458.
13	0.0	0.0	7975.
14	0.0	0.0	6039.
15	0.0	0.0	4029.
16	0.0	0.0	2510.
17	0.0	0.0	1562.
18	0.0	0.0	969.
19	0.0	0.0	599.
20	0.0	0.0	370.
21	0.0	0.0	224.
22	0.0	0.0	133.
23	0.0	0.0	72.
24	0.0	0.0	37.
25	0.0	0.0	17.
26	0.0	0.0	8.
27	0.0	0.0	2.
28	0.0	0.0	0.
29	0.0	0.0	0.
30	0.0	0.0	0.
31	0.0	0.0	0.
32	0.0	0.0	0.
33	0.0	0.0	0.
34	0.0	0.0	0.
35	0.0	0.0	0.
36	0.0	0.0	0.
37	0.0	0.0	0.
38	0.0	0.0	0.
39	0.0	0.0	0.
40	0.0	0.0	0.
41	0.0	0.0	0.
42	0.0	0.0	0.
43	0.0	0.0	0.
44	0.0	0.0	0.
45	0.0	0.0	0.
46	0.0	0.0	0.
47	0.0	0.0	0.
48	0.0	0.0	0.
SUM	14.04	14.04	85716.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	12103.	6731.	1786.	1786.	85714.
INCHES		13.24	14.05	14.05	14.05
AC-FT		3339.	3544.	3544.	3544.

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********** ********** ********** ********** **********

SUB-AREA RUNOFF COMPUTATION

THIS HYDROGRAPH IS FOR SPLIT ROCK POND WATERSHED LAKE AREA

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME
2	0	0	0	2	1	1

HYDROGRAPH DATA

IHYDG	IUHG	TAREA	SNAP	TRSDA	TRSPC	RATIO	ISNOW	ISAME	LOCAL
0	-1	0.86	0.0	0.0	0.0	0.0	0	0	0

PRECIP DATA

NP	STORM	DAJ	DAK
12	0.0	0.0	0.0

PRECIP PATTERN

0.0	0.09	0.39	0.60	0.94	1.08	3.07	3.36	1.27	1.28
0.98	0.98								

LOSS DATA

STRKR	DLTKR	RTIOL	ERAIN	STRKS	RTIOK	STRTL	CNSTL	ALSMX	RTIMP
0.0	0.0	1.00	0.0	0.0	1.00	0.0	0.0	0.0	1.00

GIVEN UNIT GRAPH, NUHGQ= 1

99 1110.

UNIT GRAPH TOTALS 1110. CFS OR 1.00 INCHES OVER THE AREA

RECESSION DATA

STRTQ= 0.0 QRCSN= 0.0 RTIOR= 1.00

END-OF-PERIOD FLOW

TIME	RAIN	EXCS	COMP Q
1	0.0	0.0	0.
2	0.09	0.09	100.
3	0.39	0.39	433.
4	0.60	0.60	666.
5	0.94	0.94	1043.
6	1.08	1.08	1199.
7	3.07	3.07	3408.
8	3.36	3.36	3730.
9	1.27	1.27	1410.
10	1.28	1.28	1421.
11	0.98	0.98	1088.
12	0.98	0.98	1088.
13	0.0	0.0	0.
14	0.0	0.0	0.
15	0.0	0.0	0.
16	0.0	0.0	0.
17	0.0	0.0	0.
18	0.0	0.0	0.
19	0.0	0.0	0.
20	0.0	0.0	0.
21	0.0	0.0	0.
22	0.0	0.0	0.
23	0.0	0.0	0.
24	0.0	0.0	0.

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26	0.0	0.0	0.
27	0.0	0.0	0.
28	0.0	0.0	0.
29	0.0	0.0	0.
30	0.0	0.0	0.
31	0.0	0.0	0.
32	0.0	0.0	0.
33	0.0	0.0	0.
34	0.0	0.0	0.
35	0.0	0.0	0.
36	0.0	0.0	0.
37	0.0	0.0	0.
38	0.0	0.0	0.
39	0.0	0.0	0.
40	0.0	0.0	0.
41	0.0	0.0	0.
42	0.0	0.0	0.
43	0.0	0.0	0.
44	0.0	0.0	0.
45	0.0	0.0	0.
46	0.0	0.0	0.
47	0.0	0.0	0.
48	0.0	0.0	0.
SUM	14.04	14.04	15586.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	3730.	1299.	325.	325.	15584.
INCHES		14.05	14.05	14.05	14.05
AC-FT		644.	644.	644.	644.

OVN

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********** ********** ********** ********** **********

COMBINE HYDROGRAPHS

THIS HYDROGRAPH IS FOR SPLIT ROCK POND WATERSHED COMBINED AREA

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME
3	2	0	0	2	1	1

SUM OF 2 HYDROGRAPHS AT 3

0.	124.	636.	1387.	2646.	3963.	7907.	11427.	12489.	13523.
12104.	10546.	7975.	6039.	4029.	2510.	1562.	969.	599.	370.
224.	133.	72.	37.	17.	8.	2.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.		

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	13523.	7930.	2110.	2110.	101298.
INCHES		13.20	14.05	14.05	14.05
AC-FT		3934.	4188.	4188.	4188.

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OVN

********** ********** ********** ********** **********

HYDROGRAPH ROUTING

THIS IS OUTFLOW HYDROGRAPH OF SPLIT ROCK POND WATERSHED AFTER ROUTING

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME
4	1	0	0	2	1	1

ROUTING DATA

QLOSS	CLOSS	AVG	IRES	ISAME
0.0	0.0	1.00	1	0

NSTPS	NSTDL	LAG	AMSKK	X	TSK	STORA
1	0	0	0.0	0.0	0.0	0.

STORAGE=	0.	561.	1143.	1745.	2367.	3010.	3673.	4350.	0.	0.
OUTFLOW=	0.	171.	514.	991.	1580.	2279.	2440.	2724.	0.	0.

TIME	EOP STOR	AVG IN	EOP OUT	
1	0.	0.	0.	
2	5.	124.	2.	
3	31.	636.	9.	
4	88.	1387.	27.	
5	195.	2646.	60.	
6	356.	3963.	108.	
7	675.	7907.	238.	
8	1132.	11427.	507.	
9	1619.	12489.	891.	
10	2132.	13523.	1357.	
11	2567.	12104.	1797.	
12	2920.	10546.	2181.	
13	3157.	7975.	2315.	
14	3310.	6039.	2352.	
15	3379.	4029.	2369.	
16	3385.	2510.	2370.	— peak elevation 240.5
17	3351.	1562.	2362.	
18	3294.	969.	2348.	
19	3222.	599.	2331.	
20	3142.	370.	2311.	
21	3056.	224.	2290.	
22	2968.	133.	2233.	
23	2880.	72.	2138.	
24	2796.	37.	2046.	
25	2714.	17.	1957.	
26	2635.	8.	1871.	
27	2559.	2.	1789.	
28	2487.	0.	1710.	
29	2418.	0.	1635.	
30	2352.	0.	1565.	
31	2288.	0.	1505.	
32	2227.	0.	1448.	
33	2169.	0.	1392.	
34	2112.	0.	1339.	
35	2058.	0.	1287.	
36	2006.	0.	1238.	
37	1956.	0.	1190.	
38	1907.	0.	1145.	
39	1861.	0.	1101.	

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40	1816.	0.	1059.
41	1773.	0.	1018.
42	1732.	0.	981.
43	1692.	0.	949.
44	1654.	0.	919.
45	1616.	0.	889.
46	1580.	0.	860.
47	1545.	0.	833.
48	1511.	0.	806.
SUM			65127.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	2370.	2300.	1357.	1357.	65127.
INCHES		3.83	9.03	9.03	9.03
AC-FT		1141.	2693.	2693.	2693.

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RUNOFF SUMMARY, AVERAGE FLOW

		PEAK	6-HOUR	24-HOUR	72-HOUR	AREA
HYDROGRAPH AT	1	12103.	6731.	1786.	1786.	4.73
HYDROGRAPH AT	2	3730.	1299.	325.	325.	0.86
2 COMBINED	3	13523.	7930.	2110.	2110.	5.59
ROUTED TO	4	2370.	2300.	1357.	1357.	5.59

30/17

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A          NEW JERSEY DAM INSPECTIONS - PHILADELPHIA DISTRICT C.O.E.
A                SPLIT ROCK POND DAM (SPLITROCK RESERVCIR)
A               1/2 PROBABLE MAXIMUM STORM BY UNIT HYDROGRAPHS
B       48              30
1        3
K        0       1                       2       1       1
1         THIS HYDROGRAPH IS FOR SPLIT ROCK WATERSHED LAND AREA
M        0      -1    4.73       0                     0.5
O       12
1        0    0.09    0.39    0.60    0.94    1.08    3.07    3.36    1.27    1.28
1     0.98    0.98
T
U       17
1      271    1084    1505    1233     767     481     297     175     112      71
1       45      29      18       9       6       2       0
X                        1
K        0       2                       2       1       1
U         THIS HYDROGRAPH IS FOR SPLIT ROCK POND WATERSHED LAKE AREA
M        0      -1     .86       0                     0.5
9       12
1        0    0.09    0.39    0.60    0.94    1.08    3.07    3.36    1.27    1.28
1     0.98    0.98
T                                                                            1
U        1
1     1110
X                        1
K        2       3                       2       1       1
1       THIS HYDROGRAPH IS FOR SPLIT ROCK POND WATERSHED COMBINED AREA
K        1       4                       2       1       1
1   105 THIS IS OUTFLOW HYDROGRAPH OF SPLIT ROCK POND WATERSHED AFTER ROUTING
Y                        1       1
1        1
2        0     561    1143    1745    2367    3010    3673    4350
3        0     171     514     991    1580    2279    2440    2724
K       99
A
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HEC-1 VERSION DATED JAN 1973

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                              NEW JERSEY DAM INSPECTIONS - PHILADELPHIA DISTRICT C.O.E.
                                  SPLIT ROCK POND DAM (SPLITROCK RESERVOIR)
                                1/2 PROBABLE MAXIMUM STORM BY UNIT HYDROGRAPHS

                                               JOB SPECIFICATION
                                NQ    NHR   NMIN   IDAY    IHR   IMIN METRC   IPLT   IPRT NSTAN
                                48      0     30      0      0      0     0      0      0     0
                                                    JOFER      NWT
                                                        3        0

           **********          **********          **********          **********          **********

                                           SUB-AREA RUNOFF COMPUTATION

                             THIS HYDROGRAPH IS FOR SPLIT ROCK WATERSHED LAND AREA

                                 ISTAQ    ICOMP    IECON    ITAPE     JPLT     JPRT    INAME
                                     1        0        0        0        2        1        1

                                                  HYDROGRAPH DATA
                   IHYDG     IUHG    TAREA     SNAP    TRSDA    TRSPC    RATIO    ISNOW    ISAME    LOCAL
                       0       -1     4.73      0.0      0.0      0.0    0.500        0        0        0

                                                    PRECIP DATA
                                           NP    STORM      DAJ      DAK
                                           12      0.0      0.0      0.0
                                                  PRECIP PATTERN
           0.0        0.09        0.39        0.60        0.94        1.08        3.07        3.36        1.27        1.26
           0.98       0.98

                                                     LOSS DATA
                 STRKR    DLTKR    RTIOL    ERAIN    STRKS    RTIOK    STRTL    CNSTL    ALSMX    RTIMP
                   0.0      0.0     1.00      0.0      0.0     1.00      0.0      0.0      0.0      0.0

                                           GIVEN UNIT GRAPH, NUHGQ=  17
           271.       1084.       1505.       1233.        767.        481.        297.        175.        112.         71.
            45.         29.         18.          9.          6.          2.          0.
                             UNIT GRAPH TOTALS    6105. CFS OR 1.00 INCHES OVER THE AREA

                                                  RECESSION DATA
                                STRTQ=      0.0       QRCSN=      0.0     RTIOR= 1.00

                                                END-OF-PERIOD FLOW
                                           TIME   RAIN   EXCS      CUMP Q
                                              1    0.0    0.0          0.
                                              2   0.09   0.09         24.
                                              3   0.39   0.39        203.
                                              4   0.60   0.60        721.
                                              5   0.94   0.94       1603.
                                              6   1.08   1.08       2765.
                                              7   3.07   3.07       4500.
                                              8   3.36   3.36       7697.
                                              9   1.27   1.27      11080.
                                             10   1.28   1.28      12103.
```

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11	0.98	0.98	11016.
12	0.98	0.98	9458.
13	0.0	0.0	7975.
14	0.0	0.0	6039.
15	0.0	0.0	4029.
16	0.0	0.0	2510.
17	0.0	0.0	1562.
18	0.0	0.0	969.
19	0.0	0.0	599.
20	0.0	0.0	370.
21	0.0	0.0	224.
22	0.0	0.0	133.
23	0.0	0.0	72.
24	0.0	0.0	37.
25	0.0	0.0	17.
26	0.0	0.0	8.
27	0.0	0.0	2.
28	0.0	0.0	0.
29	0.0	0.0	0.
30	0.0	0.0	0.
31	0.0	0.0	0.
32	0.0	0.0	0.
33	0.0	0.0	0.
34	0.0	0.0	0.
35	0.0	0.0	0.
36	0.0	0.0	0.
37	0.0	0.0	0.
38	0.0	0.0	0.
39	0.0	0.0	0.
40	0.0	0.0	0.
41	0.0	0.0	0.
42	0.0	0.0	0.
43	0.0	0.0	0.
44	0.0	0.0	0.
45	0.0	0.0	0.
46	0.0	0.0	0.
47	0.0	0.0	0.
48	0.0	0.0	0.
SUM	14.04	14.04	85716.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	12103.	6731.	1786.	1786.	85714.
INCHES		13.24	14.05	14.05	14.05
AC-FT		3339.	3544.	3544.	3544.

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OVN

RUNOFF MULTIPLIED BY 0.50

0.	12.	102.	360.	802.	1382.	2250.	3849.	5540.	6051.
5508.	4729.	3987.	3019.	2014.	1255.	781.	484.	299.	185.
112.	67.	36.	18.	9.	4.	1.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.		

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	6051.	3366.	893.	893.	42857.
INCHES		6.62	7.02	7.02	7.02
AC-FT		1670.	1772.	1772.	1772.

OVN

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********** ********** ********** ********** **********

SUB-AREA RUNOFF COMPUTATION

THIS HYDROGRAPH IS FOR SPLIT ROCK POND WATERSHED LAKE AREA

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME
2	0	0	0	2	1	1

HYDROGRAPH DATA

IHYDG	IUHG	TAREA	SNAP	TRSDA	TRSPC	RATIO	ISNOW	ISAME	LOCAL
0	-1	0.86	0.0	0.0	0.0	0.500	0	0	0

PRECIP DATA

NP	STORM	DAJ	DAK
12	0.0	0.0	0.0

PRECIP PATTERN

0.0	0.09	0.39	0.60	0.94	1.08	3.07	3.36	1.27	1.28
0.98	0.98								

LOSS DATA

STRKR	DLTKR	RTIOL	ERAIN	STRKS	RTIOK	STRTL	CNSTL	ALSMX	RTIMP
0.0	0.0	1.00	0.0	0.0	1.00	-0.0	0.0	0.0	1.00

GIVEN UNIT GRAPH, NUHGQ= 1

1110.

UNIT GRAPH TOTALS 1110. CFS OR 1.00 INCHES OVER THE AREA

RECESSION DATA

STRTQ= 0.0 QRCSN= 0.0 RTIOR= 1.00

END-OF-PERIOD FLOW

TIME	RAIN	EXCS	COMP Q
1	0.0	0.0	0.
2	0.09	0.09	100.
3	0.39	0.39	433.
4	0.60	0.60	666.
5	0.94	0.94	1043.
6	1.08	1.08	1199.
7	3.07	3.07	3408.
8	3.36	3.36	3730.
9	1.27	1.27	1410.
10	1.28	1.28	1421.
11	0.98	0.98	1088.
12	0.98	0.98	1088.
13	0.0	0.0	0.
14	0.0	0.0	0.
15	0.0	0.0	0.
16	0.0	0.0	0.
17	0.0	0.0	0.
18	0.0	0.0	0.
19	0.0	0.0	0.
20	0.0	0.0	0.
21	0.0	0.0	0.
22	0.0	0.0	0.
23	0.0	0.0	0.
24	0.0	0.0	0.

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26	0.0	0.0	0.
27	0.0	0.0	0.
28	0.0	0.0	0.
29	0.0	0.0	0.
30	0.0	0.0	0.
31	0.0	0.0	0.
32	0.0	0.0	0.
33	0.0	0.0	0.
34	0.0	0.0	0.
35	0.0	0.0	0.
36	0.0	0.0	0.
37	0.0	0.0	0.
38	0.0	0.0	0.
39	0.0	0.0	0.
40	0.0	0.0	0.
41	0.0	0.0	0.
42	0.0	0.0	0.
43	0.0	0.0	0.
44	0.0	0.0	0.
45	0.0	0.0	0.
46	0.0	0.0	0.
47	0.0	0.0	0.
48	0.0	0.0	0.
SUM	14.04	14.04	15586.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	3730.	1299.	325.	325.	15584.
INCHES		14.05	14.05	14.05	14.05
AC-FT		644.	644.	644.	644.

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NAV

RUNOFF MULTIPLIED BY 0.50

0.	50.	216.	333.	522.	599.	1704.	1865.	705.	710.
544.	544.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.		

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	1865.	649.	162.	162.	7792.
INCHES		7.02	7.02	7.02	7.02
AC-FT		322.	322.	322.	322.

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OVN

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**********     **********     **********     **********     **********

                          COMBINE HYDROGRAPHS

           THIS HYDROGRAPH IS FOR SPLIT ROCK POND WATERSHED COMBINED AREA

                ISTAQ   ICOMP   IECON   ITAPE    JPLT    JPRT   INAME
                    3       2       C       0       2       1       1

                        SUM OF 2 HYDROGRAPHS AT      3
     0.       62.      318.      693.     1323.     1982.     3954.     5713.     6245.     6762.
  6052.     5273.     3987.     3019.     2014.     1255.      781.      484.      299.      185.
   112.       67.       36.       18.        9.        4.        1.        0.        0.        0.
     0.        0.        0.        0.        0.        0.        0.        0.        0.        0.
     0.        0.        0.        0.        0.        0.        0.        0.

                    PEAK    6-HOUR    24-HOUR    72-HOUR   TOTAL VOLUME
          CFS      6762.     3965.      1055.      1055.         50649.
       INCHES                 6.60       7.02       7.02           7.02
        AC-FT                1967.      2094.      2094.          2094.
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OVN

********** ********** ********** ********** **********

HYDROGRAPH ROUTING

THIS IS OUTFLOW HYDROGRAPH OF SPLIT ROCK POND WATERSHED AFTER ROUTING

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME
4	1	0	0	2	1	1

ROUTING DATA

GLOSS	CLOSS	AVG	IRES	ISAME
0.0	0.0	1.00	1	0

NSTPS	NSTDL	LAG	AMSKK	X	TSK	STORA
1	0	0	0.0	0.0	0.0	0.

STORAGE=	0.	561.	1143.	1745.	2367.	3010.	3673.	4350.	0.	0.
OUTFLOW=	0.	171.	514.	991.	1580.	2279.	2440.	2724.	0.	0.

TIME	EOP STOR	AVG IN	EOP OUT
1	0.	0.	0.
2	3.	62.	1.
3	16.	318.	5.
4	44.	693.	13.
5	98.	1323.	30.
6	178.	1982.	54.
7	338.	3954.	103.
8	568.	5713.	175.
9	816.	6245.	321.
10	1079.	6762.	476.
11	1306.	6052.	643.
12	1494.	5273.	792.
13	1624.	3987.	895.
14	1710.	3019.	964.
15	1753.	2014.	999.
16	1764.	1255.	1009. — peak elev. 838.0
17	1754.	781.	1000.
18	1733.	484.	982.
19	1706.	299.	960.
20	1674.	185.	935.
21	1641.	112.	908.
22	1606.	67.	881.
23	1572.	36.	854.
24	1538.	18.	827.
25	1505.	9.	801.
26	1472.	4.	775.
27	1441.	1.	750.
28	1410.	0.	726.
29	1381.	0.	703.
30	1352.	0.	680.
31	1325.	0.	658.
32	1298.	0.	637.
33	1272.	0.	616.
34	1247.	0.	596.
35	1223.	0.	577.
36	1199.	0.	559.
37	1177.	0.	541.
38	1155.	0.	523.
39	1133.	0.	508.

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40	1113.	0.	496.
41	1092.	0.	484.
42	1073.	0.	472.
43	1053.	0.	461.
44	1034.	0.	450.
45	1016.	0.	439.
46	998.	0.	429.
47	981.	0.	418.
48	964.	0.	408.
SUM			27535.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	1009.	934.	574.	574.	27535.
INCHES		1.55	3.82	3.82	3.82
AC-FT		464.	1138.	1138.	1138.

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RUNOFF SUMMARY, AVERAGE FLOW

		PEAK	6-HOUR	24-HOUR	72-HOUR	AREA
HYDROGRAPH AT	1	6051.	3366.	893.	893.	4.73
HYDROGRAPH AT	2	1865.	649.	162.	162.	0.86
2 COMBINED	3	6762.	3965.	1055.	1055.	5.59
ROUTED TO	4	1009.	934.	574.	574.	5.59

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A             NEW JERSFY DAM INSPECTIONS - PHILADELPHIA DISTRICT C.O.E.
A                     SPLIT ROCK POND DAM (SPLITROCK RESERVOIR)
A                   100 YEAR FREQUENCY STORM BY UNIT HYDROGRAPHS
B       48                30
I        3
K        0       1                       2       1       1
I            THIS HYDROGRAPH IS FOR SPLIT ROCK WATERSHED LAND AREA
M        0      -1    4.73       0
O        7
I        0    0.26    0.40    2.30    0.70    0.30    0.24
T                                                       1.     .15
U       17
I      271    1084    1505    1233     767     481     297     175     112      71
I       45      29      18       9       6       2       0
X                        1
K        0       2                       2       1       1
U            THIS HYDROGRAPH IS FOR SPLIT ROCK POND WATERSHED LAKE AREA
M        0      -1     .86       0
O        7
I        0    0.26    0.40    2.30    0.70    0.30    0.24
T                                                                               1
U        1
I     1110
X                        1
K        2       3                       2       1       1
I          THIS HYDROGRAPH IS FOR SPLIT ROCK POND WATERSHED COMBINED AKEA
K        1       4                       2       1       1
I       THIS IS OUTFLOW HYDROGRAPH OF SPLIT ROCK POND WATERSHED AFTER ROUTING
Y                        1       1
I   116  1
2        0     561    1143    1745    2367    3010    3673    4350
3        0     171     514     991    1580    2279    2440    2724
K       99
A
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*****************************
HEC-1 VERSION DATED JAN 1973

*****************************
                    NEW JERSEY DAM INSPECTIONS - PHILADELPHIA DISTRICT C.O.E.
                          SPLIT ROCK POND DAM (SPLITROCK RESERVOIR)
                        100 YEAR FREQUENCY STORM BY UNIT HYDROGRAPHS

                                      JOB SPECIFICATION
                    NQ    NHR   NMIN   IDAY    IHR   IMIN METRC   IPLT   IPRT NSTAN
                    48      0     30      0      0      0     0      0      0     0
                                      JOPER      NWT
                                          3        0


      **********          **********          **********          **********          **********

                                 SUB-AREA RUNOFF COMPUTATION

                    THIS HYDROGRAPH IS FOR SPLIT ROCK WATERSHED LAND AREA

                    ISTAQ   ICOMP   IECON   ITAPE    JPLT    JPRT   INAME
                        1       0       0       0       2       1       1

                                       HYDROGRAPH DATA
            IHYDG    IUHG   TAREA    SNAP   TRSDA   TRSPC   RATIO   ISNOW   ISAME   LOCAL
                0      -1    4.73     0.0     0.0     0.0     0.0       0       0       0

                                        PRECIP DATA
                                 NP   STORM     DAJ     DAK
                                  7     0.0     0.0     0.0
                                       PRECIP PATTERN
       0.0      0.26      0.40      2.30      0.70      0.30      0.24

                                         LOSS DATA
           STRKR   DLTKR   RTIOL   ERAIN   STRKS   RTIOK   STRTL   CNSTL   ALSMX   RTIMP
             0.0     0.0    1.00     0.0     0.0    1.00    1.00    0.15     0.0     0.0

                              GIVEN UNIT GRAPH, NUHGQ=   17
      271.     1084.     1505.     1233.      767.      481.      297.      175.      112.       71.
       45.       29.       18.        9.        6.        2.        0.
                    UNIT GRAPH TOTALS   6105. CFS OR 1.00 INCHES OVER THE AREA

                                       RECESSION DATA
                    STRTQ=     0.0     QRCSN=     0.0     RTIOR= 1.00

                                    END-OF-PERIOD FLOW
                              TIME    RAIN    EXCS     COMP Q
                                 1     0.0     0.0         0.
                                 2    0.26    0.00         0.
                                 3    0.40    0.00         0.
                                 4    2.30    1.90       514.
                                 5    0.70    0.63      2225.
                                 6    0.30    0.23      3592.
                                 7    0.24    0.17      3567.
                                 8     0.0     0.0      2742.
                                 9     0.0     0.0      1917.
                                10     0.0     0.0      1240.
                                11     0.0     0.0       752.
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12	0.0	0.0	468.
13	0.0	0.0	293.
14	0.0	0.0	184.
15	0.0	0.0	118.
16	0.0	0.0	74.
17	0.0	0.0	42.
18	0.0	0.0	26.
19	0.0	0.0	13.
20	0.0	0.0	4.
21	0.0	0.0	1.
22	0.0	0.0	0.
23	0.0	0.0	0.
24	0.0	0.0	0.
25	0.0	0.0	0.
26	0.0	0.0	0.
27	0.0	0.0	0.
28	0.0	0.0	0.
29	0.0	0.0	0.
30	0.0	0.0	0.
31	0.0	0.0	0.
32	0.0	0.0	0.
33	0.0	0.0	0.
34	0.0	0.0	0.
35	0.0	0.0	0.
36	0.0	0.0	0.
37	0.0	0.0	0.
38	0.0	0.0	0.
39	0.0	0.0	0.
40	0.0	0.0	0.
41	0.0	0.0	0.
42	0.0	0.0	0.
43	0.0	0.0	0.
44	0.0	0.0	0.
45	0.C	0.0	0.
46	0.0	0.0	0.
47	0.0	0.0	0.
48	0.0	0.0	0.
SUM	4.20	2.93	17772.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	3592.	1468.	370.	370.	17772.
INCHES		2.89	2.91	2.91	2.91
AC-FT		728.	735.	735.	735.

OVN

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**********          **********          **********          **********          **********

                              SUB-AREA RUNOFF COMPUTATION

                   THIS HYDROGRAPH IS FOR SPLIT ROCK POND WATERSHED LAKE AREA

                   ISTAQ   ICOMP   IECON   ITAPE   JPLT   JPRT   INAME
                       2       0       0       0      2      1       1

                                   HYDROGRAPH DATA
   IHYDG    IUHG   TAREA    SNAP   TRSDA   TRSPC   RATIO   ISNOW   ISAME   LOCAL
       0      -1    0.86     0.0     0.0     0.0     0.0       0       0       0

                                     PRECIP DATA
                              NP   STORM     DAJ     DAK
                               7     0.0     0.0     0.0
                                   PRECIP PATTERN
   0.0       0.26      0.40      2.30      0.70      0.30      0.24

                                      LOSS DATA
   STRKR   DLTKR   RTIOL   ERAIN   STRKS   RTIOK   STRTL   CNSTL   ALSMX   RTIMP
     0.0     0.0    1.00     0.0     0.0    1.00     0.0     0.0     0.0    1.00

                              GIVEN UNIT GRAPH, NUHGQ=   1
1110.
                   UNIT GRAPH TOTALS   1110. CFS OR 1.00 INCHES OVER THE AREA

                                   RECESSION DATA
                     STRTQ=     0.0     QRCSN=     0.0     RTIOR= 1.00

                                 END-OF-PERIOD FLOW
                              TIME   RAIN   EXCS   COMP Q
                                 1    0.0    0.0       0.
                                 2   0.26   0.26     289.
                                 3   0.40   0.40     444.
                                 4   2.30   2.30    2553.
                                 5   0.70   0.70     777.
                                 6   0.30   0.30     333.
                                 7   0.24   0.24     266.
                                 8    0.0    0.0       0.
                                 9    0.0    0.0       0.
                                10    0.0    0.0       0.
                                11    0.0    0.0       0.
                                12    0.0    0.0       0.
                                13    0.0    0.0       0.
                                14    0.0    0.0       0.
                                15    0.0    0.0       0.
                                16    0.0    0.0       0.
                                17    0.0    0.0       0.
                                18    0.0    0.0       0.
                                19    0.0    0.0       0.
                                20    0.0    0.0       0.
                                21    0.0    0.0       0.
                                22    0.0    0.0       0.
                                23    0.0    0.0       0.
                                24    0.0    0.0       0.
                                25    0.0    0.0       0.
```

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27	0.0	0.0	0.
28	0.0	0.0	0.
29	0.0	0.0	0.
30	0.0	0.0	0.
31	0.0	0.0	0.
32	0.0	0.0	0.
33	0.0	0.0	0.
34	0.0	0.0	0.
35	0.0	0.0	0.
36	0.0	0.0	0.
37	0.0	0.0	0.
38	0.0	0.0	0.
39	0.0	0.0	0.
40	0.0	0.0	0.
41	0.0	0.0	0.
42	0.0	0.0	0.
43	0.0	0.0	0.
44	0.0	0.0	0.
45	0.0	0.0	0.
46	0.0	0.0	0.
47	0.0	0.0	0.
48	0.0	0.0	0.
SUM	4.20	4.20	4662.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	2553.	388.	97.	97.	4662.
INCHES		4.20	4.20	4.20	4.20
AC-FT		193.	193.	193.	193.

OVN

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COMBINE HYDROGRAPHS

THIS HYDROGRAPH IS FOR SPLIT ROCK POND WATERSHED COMBINED AREA

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME
3	2	0	0	2	1	1

SUM OF 2 HYDROGRAPHS AT 3

0.	289.	444.	3067.	3002.	3925.	3834.	2742.	1917.	1240.
752.	468.	293.	184.	118.	74.	42.	26.	13.	4.
1.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	3925.	1831.	467.	467.	22434.
INCHES		3.05	3.11	3.11	3.11
AC-FT		908.	928.	928.	928.

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HYDROGRAPH ROUTING

THIS IS OUTFLOW HYDROGRAPH OF SPLIT ROCK POND WATERSHED AFTER ROUTING

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME
4	1	0	0	2	1	1

ROUTING DATA

QLOSS	CLOSS	AVG	IRES	ISAME
0.0	0.0	1.00	1	0

NSTPS	NSTDL	LAG	AMSKK	X	TSK	STORA
1	0	0	0.0	0.0	0.0	0.

STORAGE=	0.	561.	1143.	1745.	2367.	3010.	3673.	4350.	0.	0.
OUTFLOW=	0.	171.	514.	991.	1580.	2279.	2440.	2724.	0.	0.

TIME	EOP STOR	AVG IN	EOP OUT
1	0.	0.	0.
2	12.	289.	4.
3	30.	444.	9.
4	155.	3067.	47.
5	277.	3002.	84.
6	435.	3925.	132.
7	586.	3834.	186.
8	691.	2742.	247.
9	759.	1917.	288.
10	798.	1240.	311.
11	816.	752.	321.
12	822.	468.	325. —peak elevation 836.5
13	820.	293.	324.
14	815.	184.	321.
15	806.	118.	316.
16	797.	74.	310.
17	786.	42.	303.
18	774.	26.	297.
19	763.	13.	290.
20	751.	4.	283.
21	740.	1.	276.
22	728.	0.	270.
23	717.	0.	263.
24	707.	0.	257.
25	696.	0.	251.
26	686.	0.	245.
27	676.	0.	239.
28	666.	0.	233.
29	657.	0.	227.
30	647.	0.	222.
31	638.	0.	217.
32	629.	0.	211.
33	621.	0.	206.
34	612.	0.	201.
35	604.	0.	196.
36	596.	0.	192.
37	588.	0.	187.
38	581.	0.	183.
39	573.	0.	178.

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40	566.	0.	174.
41	559.	0.	170.
42	552.	0.	168.
43	545.	0.	166.
44	538.	0.	164.
45	531.	0.	162.
46	525.	0.	160.
47	518.	0.	158.
48	512.	0.	156.
SUM			10129.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	325.	307.	211.	211.	10129.
INCHES		0.51	1.40	1.40	1.40
AC-FT		152.	419.	419.	419.

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RUNOFF SUMMARY, AVERAGE FLOW

		PEAK	6-HOUR	24-HOUR	72-HOUR	AREA
HYDROGRAPH AT	1	3592.	1468.	370.	370.	4.73
HYDROGRAPH AT	2	2553.	388.	97.	97.	0.86
2 COMBINED	3	3925.	1831.	467.	467.	5.59
ROUTED TO	4	325.	307.	211.	211.	5.59